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TECHNICAL REPORT SL-80-4

STRENGTH DESIGN OF REINFORCED CONCRETE HYDRAULIC STRUCTURES

Report 3

T-WALL DESIGN

by

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This report covers the specific strength design criteria and strength design procedures for inverted T-walls used as retaining walls or flood walls founded on earth or rock. Among the subjects covered are: applicable loads and forces; loading combinations; base reaction; design strength for reinforcement; distribution of flexural reinforcement; control of deflections; shrinkage and temperature reinforcement; concrete cover for reinforcement; details of

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20. ABSTRACT (Continued).

reinforcement; maximum tension reinforcement; minimum reinforcement of flexural members; combined flexure and axial loads; factored shear force; shear strength of walls; design of structural components; and design examples. A commentary discussing the considerations and background information used in developing the strength design criteria is also included.

PREFACE

The study reported herein was conducted in the Structures Laboratory (SL), U. S. Army Engineer Waterways Experiment Station (WES), under the sponsorship of the Office, Chief of Engineers (OCE), U. S. Army, as a part of Civil Works Investigation Work Unit 31623. Mr. Donald R. Dressler of the Structures Branch, Engineering Division, OCE, served as technical monitor.

This study was conducted during the period October 1979 to September 1980 under the general supervision of Messrs. Bryant Mather, Chief, SL, and John Scanlon, Chief, Concrete Technology Division, SL. This study was conducted and the report was prepared by Dr. Tony C. Liu, SL.

The assistance and cooperation of many persons were instrumental in the successful completion of this study. Particular thanks are due Mr. Scott Gleason of the U. S. Army Engineer District, Tulsa, for preparing design examples presented in Part IV of this report. The author also wishes to acknowledge Mr. Dressler, OCE; Professor Phil M. Ferguson, University of Texas at Austin; Mr. Ervell A. Staab, Missouri River Division; Mr. Chester F. Berryhill, Southwestern Division; Mr. V. M. Agostinelli, Lower Mississippi Valley Division; Mr. Garland E. Young, Fort Worth District; Mr. Marion M. Harter, Kansas City District; and Mr. William A. Price, Dr. Paul Mlakar, and Dr. N. Radhakrishnan, WES, for their critical review of the manuscript.

The Commanders and Directors of WES during this study and the preparation and publication of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. The Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
kips (force)	4448.222	newtons
kips (force) per foot	1459.3904	newtons per metre
kips (force) per inch	175126.8	newtons per metre
kips (force) per square foot	47.88026	kilopascals
kips (force) per square inch	6.894757	megapascals
kips (force)-feet	1355.818	newton-metres
kips (force)-inches	112.9848	newton-metres
pounds (force)	4.448222	newtons
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches	6.4516	square centimetres

STRENGTH DESIGN OF REINFORCED CONCRETE HYDRAULIC STRUCTURES

T-WALL DESIGN

PART I: INTRODUCTION

Objective

1. The development of strength design methods for reinforced concrete hydraulic structures was initiated in October 1978 at the U. S. Army Engineer Waterways Experiment Station (WES). The overall objective of this study is to develop a realistic strength design methodology for reinforced concrete hydraulic structures and to devise an accurate and efficient design procedure for implementing these strength design methods.

2. The first phase of this study was to develop preliminary strength design criteria that would yield designs (i.e., member dimensions, reinforcements, etc.) that are equivalent to those designed by the working-stress method for hydraulic structures. The results of this first phase study were published in Report 1 (Liu 1980).

3. The second phase of this study is to develop strength design methodology and practical design procedures that account for the special loading and service characteristics of retaining walls and floodwalls. The results of this phase are reported herein.

Scope

4. This report covers the specific strength design criteria and strength design procedures for inverted T-walls used as reinforced concrete retaining walls or floodwalls founded on earth or rock. Various types of inverted T-walls used as retaining walls and floodwalls are shown in Figure 1.

5. This report does not establish the requirements of structural

and foundation stability analyses. Such analyses should be performed in accordance with the appropriate engineering manuals.

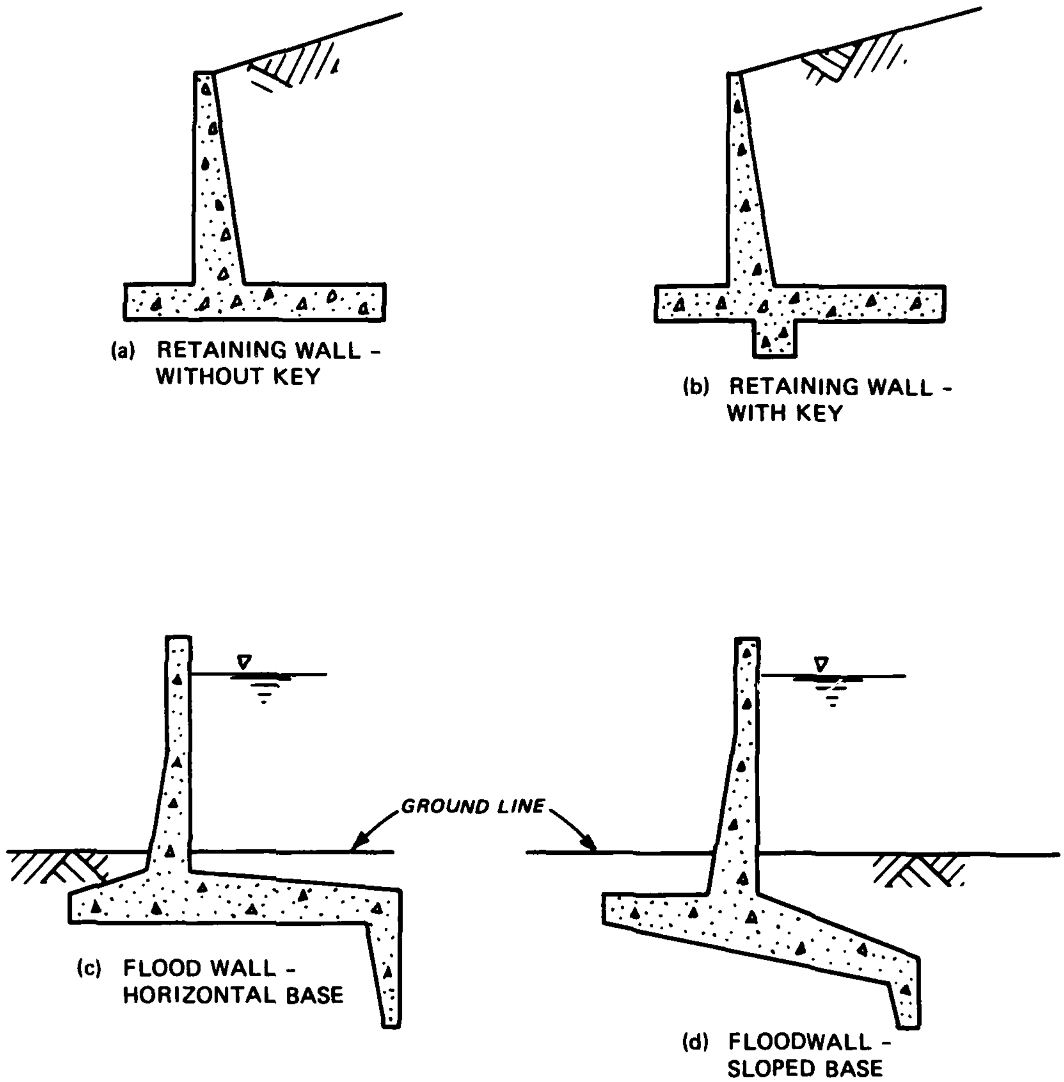

Figure 1. Typical inverted T-walls used as retaining walls and floodwalls

PART II: STRENGTH DESIGN CRITERIA

Introduction

6. The strength design criteria for inverted T-walls used as reinforced concrete retaining walls or floodwalls are defined in the following paragraphs. The considerations and background information used in developing these criteria are given in Appendix A.

General Design Criteria

7. The reinforced concrete inverted T-walls used as retaining walls or floodwalls should be designed by the strength design method in accordance with the current "Building Code Requirements for Reinforced Concrete," ACI 318-77 (American Concrete Institute 1977) except as hereinafter specified.

8. The notations used are the same as those used in ACI 318-77, except those defined in this report.

Loads and Forces

9. A typical retaining wall of floodwall should be designed to resist the following applicable loads and forces: dead load, vertical earth pressure, lateral earth pressure, vertical water pressure, lateral water and seepage pressure, uplift force, wind load, wave action, surcharge load, earthquake load, and other structural effects of differential settlement, creep, shrinkage, and temperature change.

Dead load (D)

10. The dead load should consist of the weight of the wall. The resultant of this weight acts through the center of gravity of the structure. The unit weight of reinforced concrete may be assumed to be 150 lb/ft^3* in computing the dead load.

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* A table of factors for converting inch-pound units of measurements to metric (SI) units is presented on page 4.

Vertical earth pressure (H_w)

11. The vertical earth pressure on a horizontal plane should be considered to be equal to the depth of the plane below the ground surface multiplied by the average unit weight of the soil. Because of the buoyant effect of the water on the soil particles, the weight of the submerged soil should be reduced by an amount equal to the weight of the water displaced by the soil particles.

Lateral earth pressure (H_p)

12. Active earth pressure. Walls that will rotate or translate outward during backfilling should be designed for active earth pressures. In general, inverted T-walls used as retaining walls or floodwalls on soil foundations can be designed for active pressures, whereas walls on unyielding foundations such as rock or end-bearing piles should be designed for at-rest earth pressures. However, the stem of a T-wall on an unyielding foundation may have sufficient flexibility to allow the required wall movement to occur.

13. For the purposes of design, Coulomb's theory given in EM 1110-2-2502 (U. S. Army, Office, Chief of Engineers 1961) should be used to predict the active earth pressures that are likely to occur. For more complicated multiple-layer soil systems, the incremental trial wedge method (U. S. Army, Office, Chief of Engineers 1979) may be used.

14. At-rest earth pressure. Walls which cannot move sufficiently to develop active earth pressures should be designed for at-rest earth pressures. Because the at-rest pressures can often be 50 to 100 percent greater than the corresponding active pressures for a given soil, it is important to make the appropriate choice of type of pressure for design.

15. The design guidance given in EM 1110-2-2502 may be used for estimating at-rest earth pressures for design purposes. Other methods may also be used when appropriate.

16. Passive earth pressures. The passive earth pressures should be calculated in accordance with EM 1110-2-2502. However, because of its uncertain nature, passive earth pressures should be disregarded when

other more positive means of resistance are available.

17. Location of resultant lateral pressure. Unless special conditions warrant a detailed and comprehensive analysis of the wall, the locations of resultant lateral earth pressure specified in EM 1110-2-2502 should be used.

Vertical water pressure (F_w)

18. The vertical water pressure on a horizontal plane should be considered to be equal to the depth of the plane below the water surface multiplied by 62.5 lb/ft^3.

Lateral water and seepage pressure (F_p)

19. For a retaining wall with a permeable backfill and no drainage system provided in the back of the wall, full hydrostatic pressure should be used. In the case of a floodwall or a retaining wall with a drainage system, the lateral water pressure should be determined by either creep method or flow net method as described in EM 1110-2-2501 (U. S. Army, Office, Chief of Engineers 1948).

20. Water may enter cracks that develop in cohesive soil from the surface downward due to skrinkage and stress conditions. Full hydrostatic pressure should be used if such water is adjacent to the back of a wall.

Uplift (F_u)

21. The uplift should be assumed to act over 100 percent of the base area. For walls on rock, the uplift should be assumed to vary uniformly from the water pressure at the heel to that at the toe of the wall. Uplift pressures for walls resting on soil should be determined by creep method or flow net method. Uplift pressure should be computed assuming no creep loss for the portion of the foundation not in compression.

Wind load (W)

22. When retaining walls are constructed in an exposed location, wind loads should be considered during construction and prior to

backfill. For floodwalls, wind loads should be considered throughout the entire life of the structure. The wind load forces per square foot of the exposed area specified in EM 1110-2-2502 and EM 1110-2-2501 should be used for retaining walls and floodwalls, respectively.

Wave action (p)

23. Floodwalls that are subjected to the wave action should be designed to resist the forces induced thereby. The wave forces on a wall may be calculated in accordance with U. S. Army Coastal Engineering Research Center (1973).

Surcharge loads (S_L)

24. A retaining wall may be required to carry additional loads on the surface of the backfill. The effect of these surcharge loads should be taken into account in accordance with EM 1110-2-2502.

Earthquake loads (E)

25. When a wall is to be built in a region where an earthquake may be anticipated, earthquake loads should be provided for in the design. The seismic coefficient method as described in EM 1110-2-2502 should generally be used for estimating earthquake forces.

26. When a wall is used as part of a dam and failure of the wall could result in loss of life or extensive property damage, the earthquake forces should be determined in accordance with ER 1110-2-1806 (U. S. Army, Office, Chief of Engineers 1977).

Other structural effects (T)

27. When structural effects of differential settlement, creep, shrinkage, or temperature change may be significant, they should be considered in the design. Estimations of differential settlement, creep, shrinkage, or temperature change should be based on a realistic assessment of such effect occurring in service.

Strength Requirement

Required strength

28. Reinforced concrete retaining walls or floodwalls should be

designed to have design strengths in all sections at least equal to the required strengths calculated for the factored loads and forces in the following combinations that are applicable:

$$U = 1.5D + 1.9(H_w + H_p + F_w + F_p + F_u + S_L)^* \quad (1)$$

$$U = 0.9D + 1.9(H_w + H_p + F_w + F_p + F_u) \quad (2)$$

$$U = 0.9D + 1.9W \quad (3)$$

$$U = 0.75[1.5D + 1.9(H_w + H_p + F_w + F_p + F_u + S_L + W + P)] \quad (4)$$

$$U = 0.75[1.5D + 1.9(H_w + H_p + F_w + F_p + F_u + S_L + E)] \quad (5)$$

$$U = 0.75[1.5(D + T) + 1.9(H_w + H_p + F_w + F_p + F_u + S_L)] \quad (6)$$

Base reaction

29. For strength design of wall footings, the base reactions induced by the applied "factored" loads should be used.

30. The stability analyses should be performed based on "unfactored" loads.

Design strength for reinforcement

31. Except for calculating development length, design should be based on yield strengths of reinforcement of 40,000 and 48,000 psi for for ASTM Grade 40 and Grade 60 steels, respectively. The reinforcement with yield strength in excess of Grade 60 should not be used, except for prestressing tendons. The yield strength of reinforcement of 40,000 and 60,000 psi should be used for calculating development length for ASTM Grade 40 and Grade 60 steels, respectively.

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* For a wall with a key, alternate earth pressure distributions as defined in paragraph S-21 of EM 1110-2-2501 should be investigated for base and key designs.

Serviceability Requirement

Distribution of flexural reinforcement

32. The spacing of flexural tension reinforcement generally should not exceed 18 in. for Grade 40 steels. When design yield strength for tension reinforcement exceeds 40,000 psi, the spacing of flexural tension reinforcement generally should not exceed 12 in.

33. The spacing of flexural tension reinforcement exceeding the limit specified in paragraph 32 may be used if it can be justified in terms of flexural cracking and serviceability requirements.

Control of deflections

34. Deflections at service loads need not be computed if the limit of reinforcement ratio specified in paragraph 39 is not exceeded.

35. For a reinforcement ratio exceeding the limit specified in paragraph 39, deflections at service loads should be computed in accordance with ACI 318-77, or other methods that predict deflections in substantial agreement with the results of comprehensive tests.

Shrinkage and temperature reinforcement

36. The area of shrinkage and temperature reinforcement in each direction should not be less than 0.1 percent of the gross cross-sectional area up to a maximum of No. 6 bars spaced 12 in. center-to-center in the exposed face, and only horizontal spacer bars in the backfilled face are required. Where temperature variations are extreme or where there is unusual restraint of the wall against horizontal shrinkage, the reinforcement requirement specified in EM 1110-2-2103 (U. S. Army, Office, Chief of Engineers 1971) should be followed.

Concrete cover for reinforcement

37. The following minimum concrete clear cover should be provided for principal reinforcement.

Location	Minimum Clear Cover, in.
Stem	3
Base, top face	3
Base, bottom face	4
Key	3

Details of reinforcement

38. Bending and splicing of reinforcement and minimum reinforcement spacing for walls should be in accordance with the requirements specified in EM 1110-2-2103.

Flexure and Axial Loads

Maximum tension reinforcement

39. For flexural members, and for members subjected to combined flexure and compressive axial load when the design axial load strength ϕP_n is less than $0.10f'_cA_g$, the ratio of tension reinforcement ρ generally should not exceed $0.25\rho_b$, where ρ_b is the reinforcement ratio producing balanced strain conditions as specified in ACI 318-77.

40. Reinforcement ratios exceeding the limits specified in paragraph 39 but less than $0.50\rho_b$ may be used if deflections are shown not to exceed the limit given in paragraph 35.

41. Reinforcement ratios in excess of $0.50\rho_b$ shall not be used unless a detailed investigation of serviceability requirements, including computation of deflections, is conducted in consultation with higher authority.

Minimum reinforcement of flexural members

42. At any section of a wall where tension reinforcement is required by analysis, the minimum reinforcement requirements specified in ACI 318-77 shall apply except that the f_y shall be in accordance with paragraph 31.

Combined flexure and axial loads

43. Stems subjected to small compressive loads caused by the weight of the concrete may be designed for the maximum factored moment

disregarding the axial load. For a wall with a key, the base slab generally is subjected to a combined flexure and axial load and should be designed in accordance with the following:

a. Design assumptions and general requirements.

(1) Maximum usable strain ε_C at extreme concrete compression fiber is assumed equal to 0.003. The allowable strain ε_M at extreme concrete compression fiber should be limited to $0.5\varepsilon_C$ for hydraulic structures.

(2) Balanced conditions exist at a cross section when the tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed allowable strain ε_M .

(3) Concrete stress of $0.85 f'_c$ should be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_M C$ from the fiber of maximum compressive strain.

(4) Factor β_M should be taken as 0.55 for concrete strengths f'_c up to and including 4000 psi. For strengths above 4000 psi, β_M should be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but β_M should not be taken less than 0.50.

(5) The eccentricity ratio e'/d should be defined as

$$\frac{e'}{d} = \frac{\frac{M_u}{P_u} + \left(d - \frac{h}{2}\right)}{d} \tag{7}$$

where

e' is the eccentricity of axial load measured from the centroid of the tension reinforcement

P_u is considered positive for compression and negative for tension

b. Flexural and compression capacity--tension reinforcement only.

(1) The design axial load strength ϕP_n of compression members should not be taken greater than the following:

(a) Grade 40 steel

$$\phi P_{N(max)} = 0.70\ \phi[0.85\ f'_c(A_g - \rho bd) + f_y \rho bd)] \tag{8a}$$

(b) Grade 60 steel

$$\phi P_{N(max)} = 0.70\ \phi[0.85\ f'_c(A_g - \rho bd) + E_s \varepsilon_M \rho bd] \tag{8b}$$

(2) The strength of a cross section is controlled by compression if the load has an eccentricity ratio e'/d no greater than that given by Equation 9 and by tension if e'/d exceeds this value.

$$\frac{e'_b}{d} = \frac{2k_b - k_b^2}{2k_b - \dfrac{f_y \rho}{0.425\ f'_c}} \tag{9}$$

where

$$k_b = \frac{B_M E_s \varepsilon_M}{E_s \varepsilon_M + f_y} \tag{10}$$

(3) Sections controlled by tension should be computed by

$$\phi P_n = \phi(0.85\ f'_c k_u bd - f_y \rho bd) \tag{11}$$

and

$$\phi M_n = \phi(0.85\ f'_c k_u bd - f_y \rho bd)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right]d \tag{12}$$

where k_u should be determined from the following equation:

$$k_u^2 + 2\left(\frac{e'}{d} - 1\right)k_u - \frac{f_y \rho e'}{0.425\ f'_c d} = 0 \tag{13}$$

(4) Sections controlled by compression should be computed by

$$\phi P_n = \phi(0.85\ f'_c k_u bd - f_{su} \rho bd) \qquad (14)$$

and

$$\phi M_n = \phi(0.85\ f'_c k_u bd - f_{su} \rho bd) \left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] d \qquad (15)$$

where

$$f_{su} = \frac{E_s \varepsilon_M (\beta_M - k_u)}{k_u} \qquad (16)$$

and k_u should be determined from the following equation:

$$k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2 + \left(\frac{E_s \varepsilon_M \rho e'}{0.425\ f'_c d}\right) k_u - \left(\frac{\beta_M E_s \varepsilon_M}{0.425\ f'_c}\right)\left(\frac{\rho e'}{d}\right) = 0 \qquad (17)$$

(5) The balanced load and moment should be computed using Equations 11 and 12 with $k_u = k_b$ and $e'/d = e'_b/d$. The e'_b/d and k_b are given by Equations 9 and 10, respectively.

c. Flexural and compression capacity--tension and compression reinforcement.*

(1) The design axial load strength ϕP_n of compression members should not be taken greater than the following:

(a) Grade 40 steel

$$\phi P_{n(max)} = 0.70\ \phi\{0.85\ f'_c\ [A_g - (\rho + \rho')]bd + f_y(\rho + \rho')bd\} \qquad (18a)$$

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* Ties and stirrups should be provided where compression reinforcement is used.

(b) Grade 60 steel

$$\phi P_{n(max)} = 0.70\ \phi\{0.85\ f'_c\ [A_g - (\rho + \rho')]bd + E_s \varepsilon_M (\rho + \rho')bd\} \tag{18b}$$

(2) The strength of a cross section is controlled by compression if the load has an eccentricity ratio e'/d no greater than that given by Equation 19 and by tension if e'/d exceeds this value

$$\frac{e'_b}{d} = \frac{2k_b - k_b^2 + \dfrac{f'_{su}\rho'\left(1 - \dfrac{d'}{d}\right)}{0.425\ f'_c}}{2k_b - \dfrac{f_y \rho}{0.425\ f'_c} + \dfrac{f'_{su}\rho'}{0.425\ f'_c}} \tag{19}$$

where k_b is given in Equation 10 and f'_{su} is given in Equation 23 with $k_u = k_b$.

(3) Sections controlled by tension are computed by Equations 20 and 23 with $f_{su} = f_y$.

(4) Sections controlled by compression should be computed by

$$\phi P_n = \phi(0.85\ f'_c k_u bd + f'_{su}\rho' bd - f_{su}\rho bd) \tag{20}$$

and

$$\phi M_n = \phi(0.85\ f'_c k_u bd + f'_{su}\rho' bd - f_{su}\rho bd)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] d \tag{21}$$

where

$$f_{su} = \frac{E_s \varepsilon_M (\beta_M - k_u)}{k_u} \geq -f_y \tag{22}$$

$$f'_{su} = \frac{E_s \varepsilon_M \left[k_u - \beta_M\left(\frac{d'}{d}\right)\right]}{k_u} \leq f_y \qquad (23)$$

and k_u should be determined from the following equation:

$$k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2$$
$$+ \frac{E_s \varepsilon_M}{0.425\, f'_c} \left[(\rho + \rho')\left(\frac{e'}{d}\right) - \rho'\left(1 - \frac{d'}{d}\right)\right] k_u \qquad (24)$$
$$- \frac{\beta_M E_s \varepsilon_M}{0.425\, f'_c} \left[\rho\left(\frac{d'}{d}\right)\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \rho\left(\frac{e'}{d}\right)\right] = 0$$

(5) The balanced load and moment should be computed using Equations 20-23 with $k_u = k_b$ and $e'/d = e'_b/d$. The e'_b/d and k_b are given by Equations 19 and 10, respectively.

d. Flexural and tension capacity.

(1) If the load has an eccentricity ratio $e'/d < 0$, the ϕP_n and ϕM_n should be computed by Equations 11-13 disregarding the compression reinforcement, if any.

(2) If the load has an eccentricity ratio $1 - (h/2d) \geq e'/d \geq 0$, reinforcement should be provided in both faces of the member in the following amount.

$$A_s = \frac{P_u(d - d' - e')}{\phi f_y(d - d')} \qquad (25)$$

and

$$A'_s = \frac{P_u e'}{\phi f_y(d - d')} \qquad (26)$$

Shear Strength Requirements

Factored shear force

44. The maximum factored shear force should be computed at a distance d from the base of the stem for stem design, at a distance d from the stem for toe design, at the face of the stem for heel design, and at the base of the key for key design.

45. For L-walls without toes, the maximum factored shear force should be computed at the base of the stem for stem design.

Shear strength of walls

46. The shear strength of walls should be designed in accordance with ACI 318-77.

47. For keys or toes with a length-to-depth ratio of unit or less, they should be designed as brackets. The special provisions for brackets specified in ACI 318-77 should be followed.

PART III: STRUCTURAL DESIGN

General

48. After the tentative wall dimensions have been proved satisfactory with respect to stability criteria specified in the appropriate engineering manuals, the wall should be designed to provide adequate structural strength using the strength design method specified in Part II. For this purpose, the actual loads and pressures should be multiplied by the appropriate load factors, and the resulting base reaction and other resistances should be evaluated from these factored loading conditions. After the factored loadings and corresponding reactions have been determined, each component of the wall (i.e., stem, toe, heel, or key) should be designed independently as a cantilever section carrying all applied factored loads and reactions.

Design of Structural Components

Stem design

49. The stem should be designed as a vertical cantilever fixed at the top of the base. The principal force acting on the stem is generally the lateral earth and water pressures. Thus, main reinforcement should be placed at the back of the stem.

50. To facilitate concrete placement, the minimum top thickness of the stem specified in EM 1110-2-2502 and EM 1110-2-2501 should be followed for retaining walls and floodwalls, respectively.

Heel design

51. The heel should be designed as a cantilever fixed at the center of the longitudinal stem reinforcement. The load on the heel is predominantly a downward load due to the weight of backfill or water on the heel, and the reinforcement should be placed at the top of the heel across the critical section. If the key is under the heel, the heel should be designed for flexure, shear, and axial tension caused by passive pressure on the key.

Toe design

52. The toe should be designed as a cantilever fixed at the face of the stem. The principal loading on the toe is the upward force due to base reaction. The effect of axial compression due to horizontal active earth pressure at the toe is generally small and may be neglected. However, the horizontal passive earth pressure, if any, should not be neglected.

Key design

53. A vertical key extending downward from the bottom of the heel at the extreme end is generally required for floodwall design. It has the dual function of reducing the uplift and increasing the resistance against sliding. The key should be designed as a cantilever fixed at the bottom of the base. The principal loading is the passive earth pressure at the landside face of the key.

54. For retaining walls, a key should be required only if the friction resistance between the bottom of the base and the soil is not sufficient to prevent the wall from sliding. In general, only a limited amount of depth into the soil is required.

Reinforcement in base and key

55. For a wall with a key, alternate loading criteria specified in paragraph S-21 of EM 1110-2-2501 should be checked in determining reinforcement in the base and key.

Stopping reinforcement in stem

56. The bending moment in the stem decreases rapidly with increasing distance from the base. For this reason, only part of the main reinforcement is needed at higher elevations. The bar spacing and cutoff points should be determined by using the design moment curve and in accordance with the reinforcement development requirement of ACI 318-77. Detailed procedure is given in the design examples presented in Part IV.

PART IV: DESIGN EXAMPLES

Retaining Wall Design

Design data

57. The retaining wall design data are:

Overall height = 25.00 ft

Unit weight of soil (moist) = 120 lb/ft^3

Unit weight of soil (submerged) = 57.5 lb/ft^3

Friction angle, soil-on-soil, $\phi = 30°$

Cohesion, $C = 0$

Backfill surface slope, $\beta = +14°$

Saturation level (heel side), 13 ft above base

Top of ground and saturation (toe side), 5 ft above base

Specified concrete strength, $f'_c = 3000$ psi

Specified steel yield strength, $f_y = 40{,}000$ psi

Service loads and structural dimensions

58. The service loads and structural dimensions resulting from a stability analysis for one load case are shown in Figures 2 and 3, respectively. The lateral earth pressure was determined by a wedge analysis while lateral water pressure and uplift were determined by the creep method. Lateral earth pressure on the toe side was assumed to be at rest pressure. A coefficient of 0.4 was used for at rest pressure.

Factored loads

59. The loading diagrams and base reaction for the following factored loads are shown in Figure 4.

$$U = 1.5D + 1.9(H_W + H_P + F_W + F_P + F_U)$$

The moments and shears used in the strength design were obtained from these diagrams and are shown in Figures 5-7.

Stem design

60. The factored moments and shears for the stem are shown in

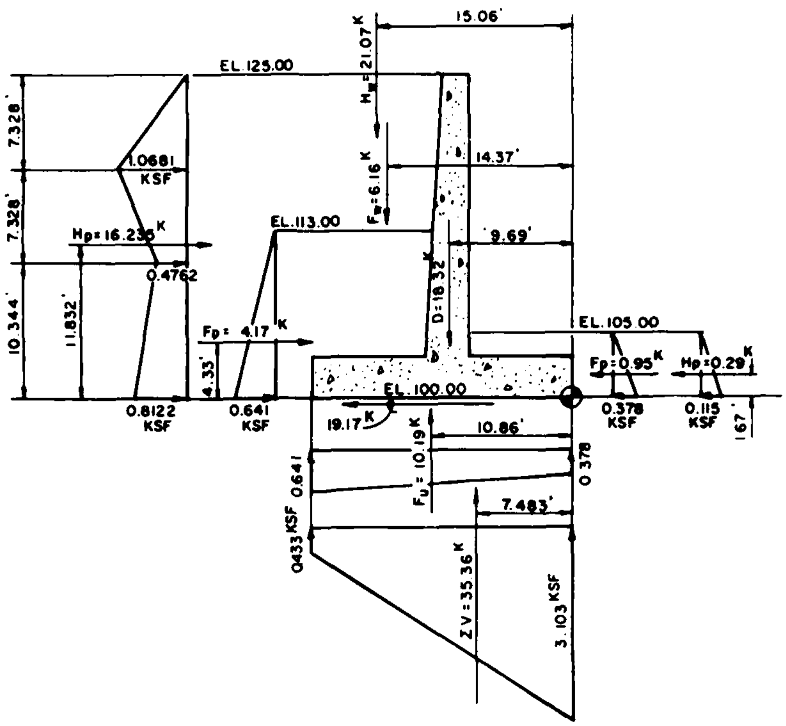


Figure 2. Load diagram - service loads

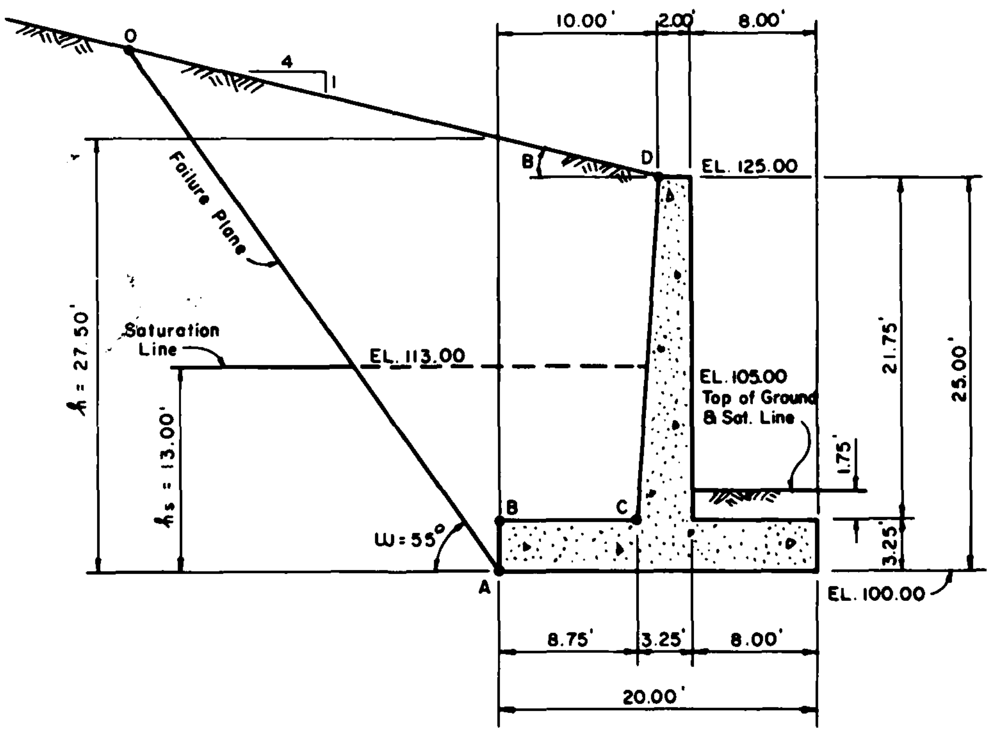


Figure 3. Structural dimensions

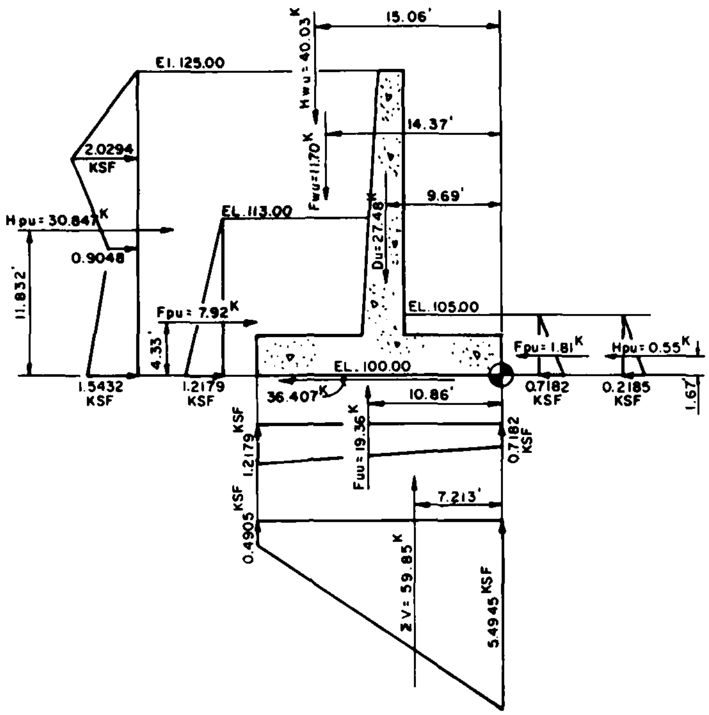


Figure 4. Loading diagram for factored loads

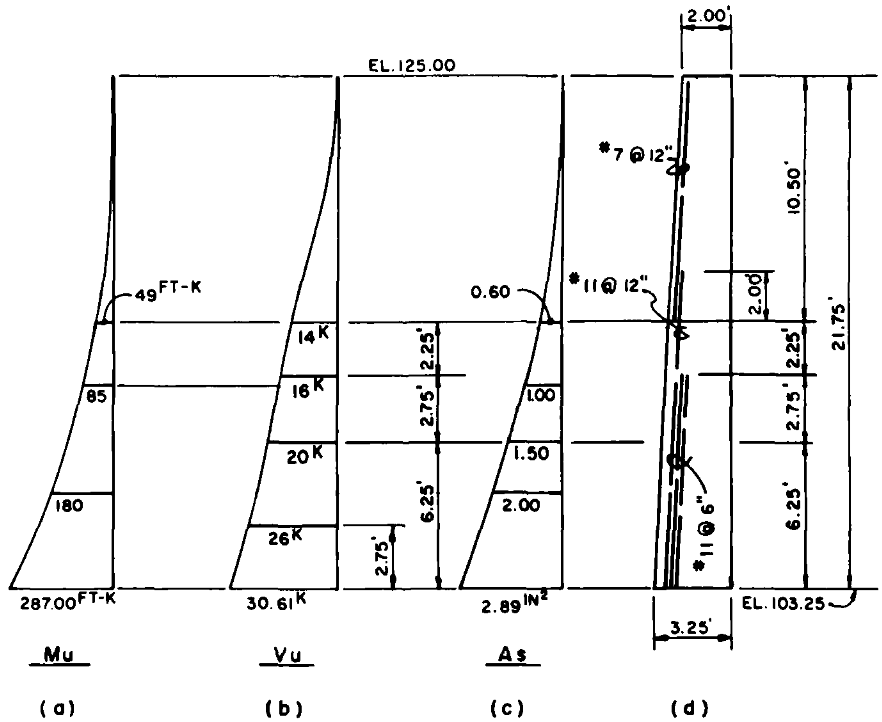


Figure 5. Stem - moments, shears, and reinforcement

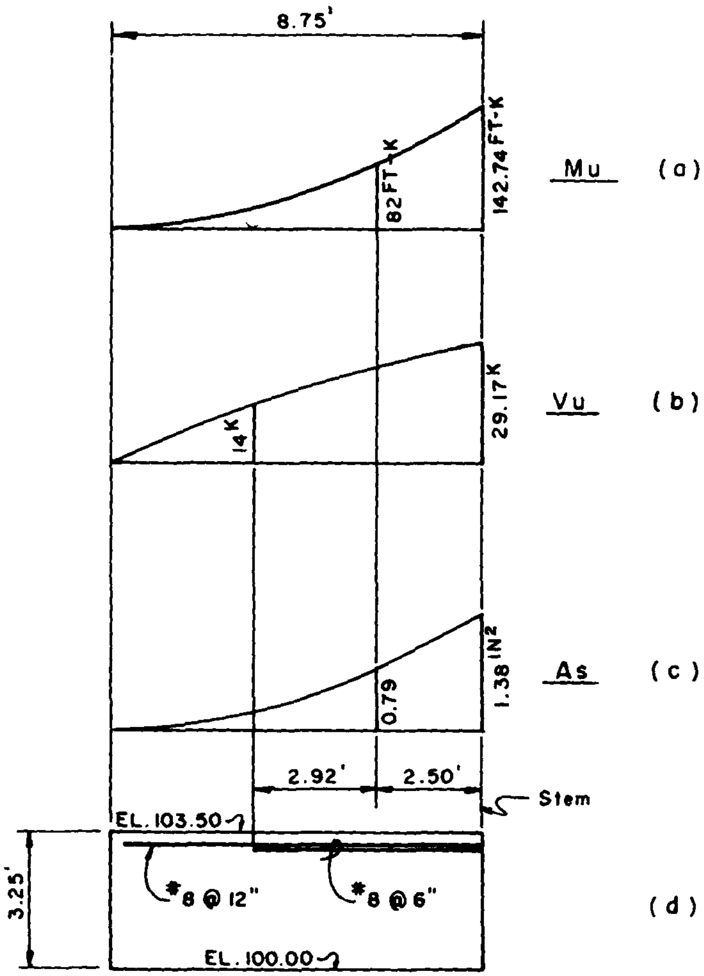


Figure 6. Heel - moments, shears, and reinforcement

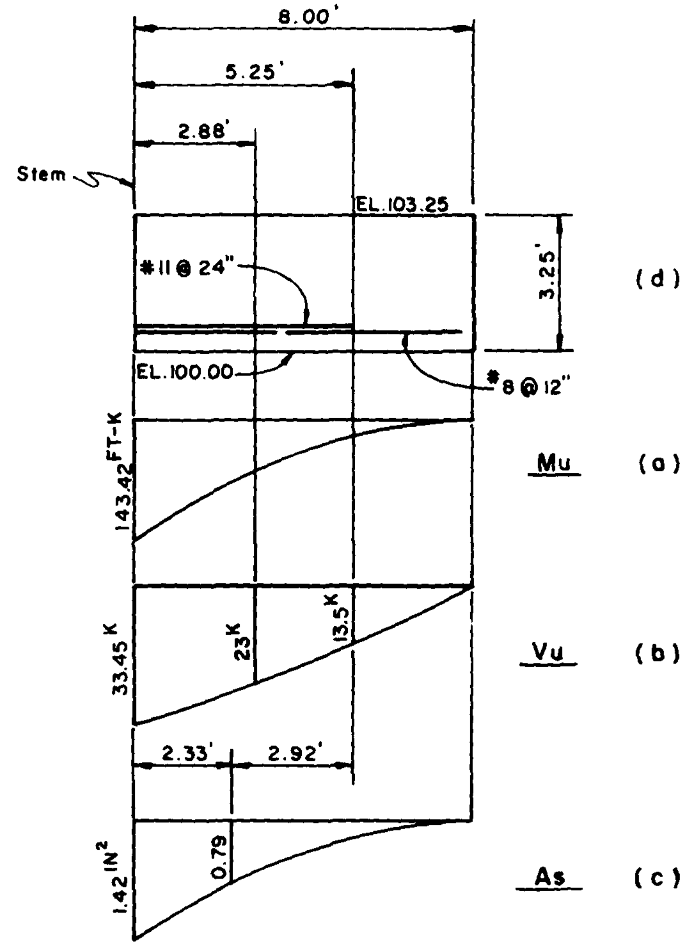


Figure 7. Toe - moments, shears, and reinforcement

Figure 5. The reinforcement required at the base of the stem can be determined as follows:

$$M_u = \phi\left[A_s f_y\left(d - \frac{A_s f_y}{2 \times 0.85\ f'_c b}\right)\right] \tag{27}$$

The substitution of

$M_u = 287 \times 12 = 3444$ in.-K (from Figure 5)

$\phi = 0.9$

$b = 12$ in.

$d = 35$ in.

$f'_c = 3$ ksi

$f_y = 40$ ksi

into Equation 27 and solving for A_s results in $A_s = 2.89$ in.^{2}. No. 11 bars on 6-in. centers will be used, giving $A_s = 3.12$ in.^{2}. Check $\rho = 3.12/(12 \times 35) = 0.0074 < 0.25\ \rho_b$.

61. It can be seen in Figure 5(c) that one-half of the No. 11 bars are no longer required at a point 6.25 ft above el 103.25. According to ACI 318-77, paragraph 12.11.3, the reinforcement shall extend a distance, equal to the effective depth of the member, past the point where it is no longer required. The effective depth 6.25 ft above el 103.25* is approximately 2.64 ft; these bars will then be discontinued at 9.00 ft above el 103.25. A check of the shear at the cutoff point is made in accordance with ACI 318-77, paragraph 12.11.5.1 as follows:

$$V_u = 16 \text{ K (Figure 5b)}$$

$$d = 35 \text{ in.} - \left(\frac{9.0}{21.75}\right)15 \text{ in.} = 28.79 \text{ in.}$$

$$b = 12 \text{ in.},\quad \phi = 0.85\ ,\quad f'_c = 3000 \text{ psi}$$

$$\phi V_c = \phi(2\sqrt{f'_c})bd = 32{,}169 \text{ lb}$$

$$2/3(\phi V_c) = 21.45\text{K} > V_u$$

---

* All elevations (el) cited herein are in feet referred to mean sea level (msl).

62. The required reinforcement and cutoffs for other points on the stem are determined in the same manner.

63. No. 11 bars at 12 in. are no longer required at el 114.50. The distance these bars must extend above el 114.50 for proper lap with No. 7 bars is determined according to ACI 318-77 as follows:

a. Basic Development Length $= \dfrac{0.04\ A_b f_y}{\sqrt{f'_c}}$ (paragraph 12.2.2)

$$= \frac{0.04 \times 0.6 \times 40{,}000}{\sqrt{3000}} = 17.53 \text{ in.}$$

b. Modification Factor = 0.8 ℓ_d (paragraph 12.2.4)

$$= 0.8 \times 17.53 = 14.02 \text{ in.}$$

c. Lap Length = 1.7 ℓ_d (paragraphs 12.16.1 and 12.16.2)

$$= 1.7 \times 14.02 = 23.83 \text{ in. (24 in.)}$$

64. The shear should be checked at a distance $d = 2.75$ ft above el 103.25 as follows:

$$V_c = 2\sqrt{f'_c}\ bd = 2\sqrt{3000} \times 12 \times 2.75 \times 12 = 43{,}380 \text{ lb}$$

$$V_u = 26 \text{ K (Figure 5b)}$$

$$\phi V_c = 36.87 \text{ K} > V_u$$

Heel design

65. The factored moments and shears for the heel are shown in Figure 6. The reinforcement in the top face of the stem should be determined for the moment existing at the point where the reinforcement in the back face of the stem intersects the base slab as follows:

$b = 12$ in., $d = 35.5$ in., M_u (at face of stem) = 142.74 ft-K (Figure 6)

$V_u = 29.17$ K (Figure 6b)

$$M_u \text{ (at design point)} = 142.74 + 29.17\left(\frac{4}{12}\right) = 152.46 \text{ ft-K}$$

$$= 1829.52 \text{ in.-K}$$

$f'_c = 3$ KS , $f_y = 40$ ksi, $\phi = 0.9$

Substitute these values into Equation 27 and solve for $A_s = 1.47 \text{ in.}^2$. No. 8 bars on 6-in. centers will be used given $A_s = 1.58 \text{ in.}^2$. Check $\rho = 1.58/(12 \times 35.5) = 0.0037 < 0.25\ \rho_b$.

66. The cutoff point for one-half of the bars was determined in the same manner as the cutoff points in the stem.

67. The shear at the face of the stem will be checked as follows:

$$V_u = 29.17 \text{ K (Figure 6b)}\ , \quad \phi = 0.85$$

$$V_c = 2\sqrt{f'_c}\ bd = 2\sqrt{3000} \times 12 \times 35.5 = 46{,}670 \text{ lb}$$

$$\phi V_c = 0.85 \times 46.67 = 39.67 \text{ K} > V_u$$

Toe design

68. The factored moments and shears for the toe are shown in Figure 7. The reinforcement required in the bottom face at the stem should be determined as follows:

$$b = 12 \text{ in.},\quad d = 34.5 \text{ in.},\quad M_u = 143.42 \text{ ft-K} = 1721.04 \text{ in.-K (Figure 7)}$$

$$f'_c = 3 \text{ ksi},\quad f_y = 40 \text{ ksi}$$

69. Substitute into Equation 27 and solve for $A_s = 1.42 \text{ in.}^2$. No. 8 bars on 12-in. centers with No. 11 bars on 24-in. centers will be used, giving $A_s = 1.57 \text{ in.}^2$. Check $\rho = 1.57/(12 \times 34.5) = 0.0038 < 0.25\ \rho_b$.

70. The cutoff point for one-half of the bars was determined in the same manner as cutoff points in the stem and heel.

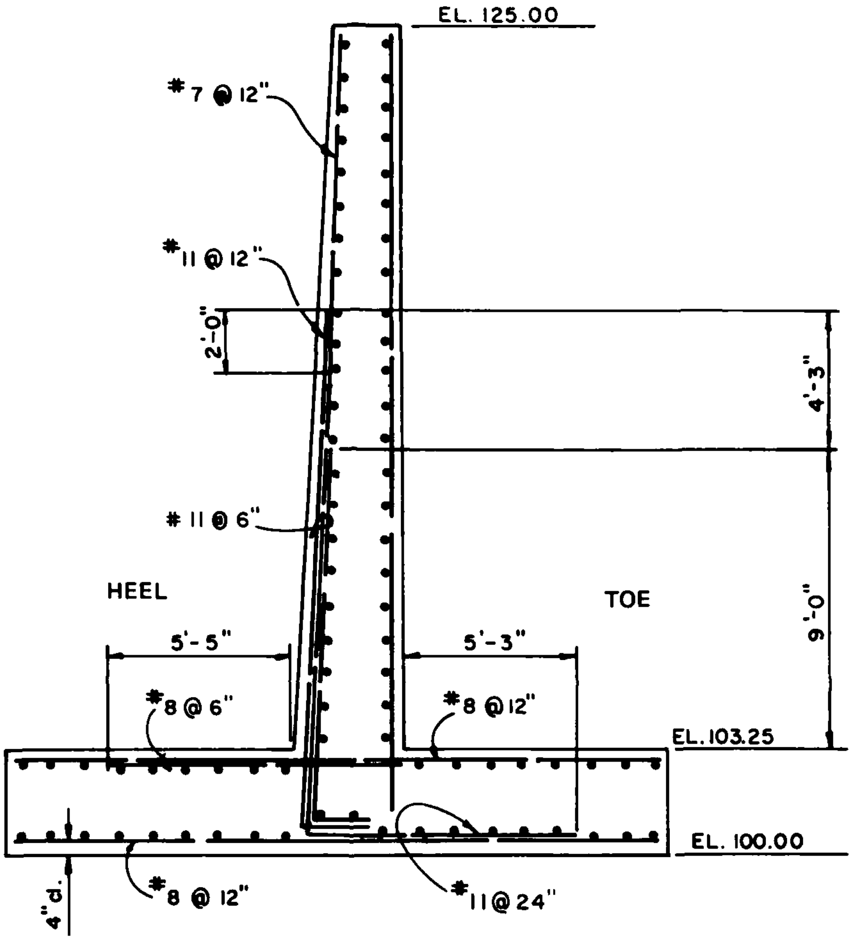
Reinforcement summary

71. A summary of the reinforcement for the entire wall is shown in Figure 8.

Floodwall Design

Design data

72. The floodwall design data are as follows:



NOTES:

1. All reinf. to be #6 @ 12", except as shown.
2. All hooks shown are ACI standard hooks.
3. Minimum clear concrete cover to be 3" except as shown.

Figure 8. Reinforcement summary

Overall height = 26.25 ft

Unit weight of soil (saturated) = 120 lb/ft^3

Unit weight of soil (submerged) = 57.5 lb/ft^3

Friction angle, soil-on-soil, $\phi = 37°$

Cohesion, $C = 0$

Active earth pressure (submerged) = 14.3 lb/ft^3

Passive earth pressure (submerged) = 231.3 lb/ft^3

Specified concrete strength, f'_c = 3000 psi

Specified steel yield strength, f_y = 40,000 psi

Structural dimensions

73. The structural dimensions and the stability analysis for the service loads are shown in Figure 9. The lateral water pressure and uplift were determined by the creep method.

Factored loads

74. The loading diagrams and the base reaction for the following factored loads are shown in Figure 10.

$$U = 1.5D + 1.9(H_W + H_P + F_W + F_P + F_U)$$

The moments, thrusts, and shears used in strength design are obtained from these loading diagrams, and are shown in Figures 11-14.

Stem design

75. The factored moments and shears for the stem are shown in Figure 11a and b. The required reinforcement at maximum moment, which occurs at the base of the stem (el 110.75), can be determined from the following equation:

$$M_u = \phi\left[A_s f_y\left(d - \frac{A_s f_y}{2 \times 0.85\ f'_c b}\right)\right] \qquad (27bis)$$

Substituting

M_u = 228.18 × 12 = 2738.2 in.-K

ϕ = 0.9

b = 12 in.

d = 29 in.

f'_c = 3 ksi

f_y = 40 ksi

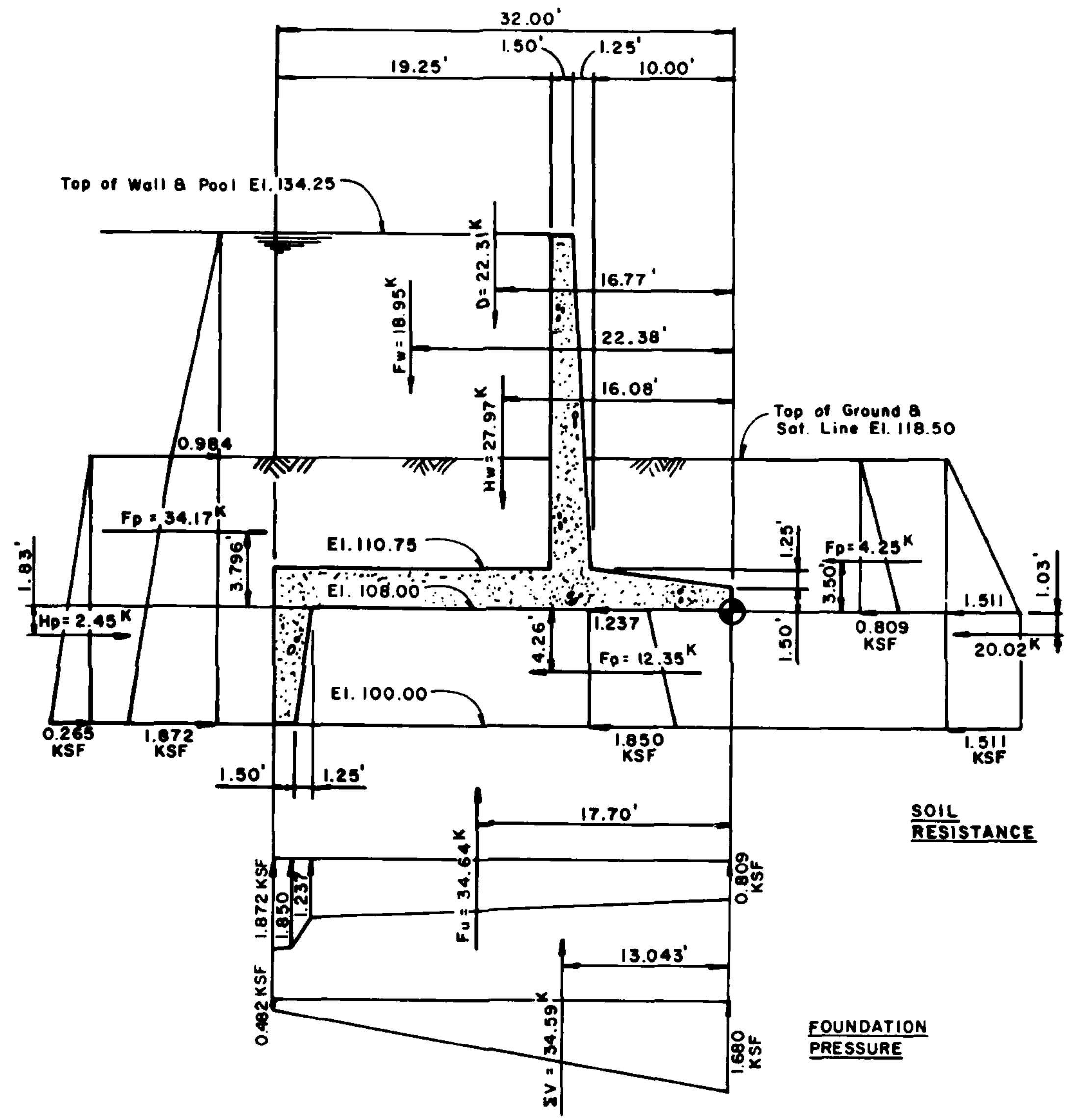


Figure 9. Dimensions and service loads

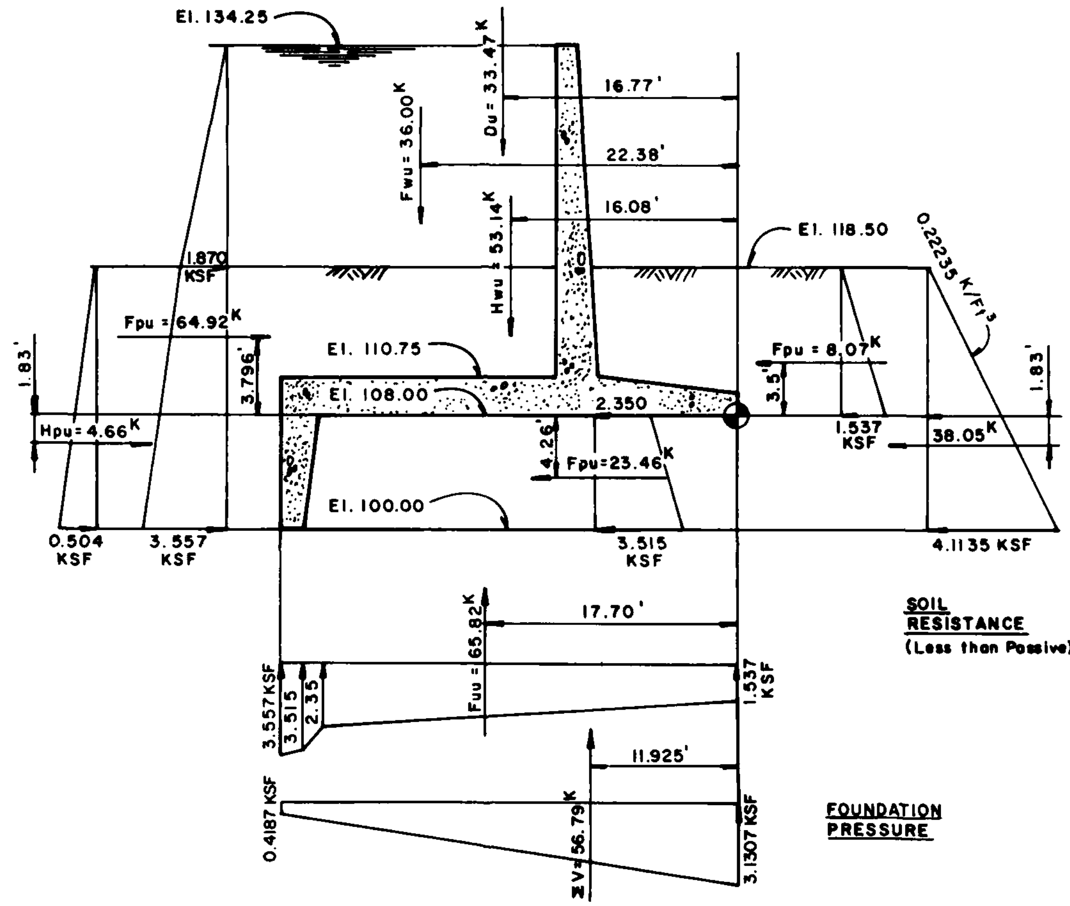


Figure 10. Loading diagram for factored loads

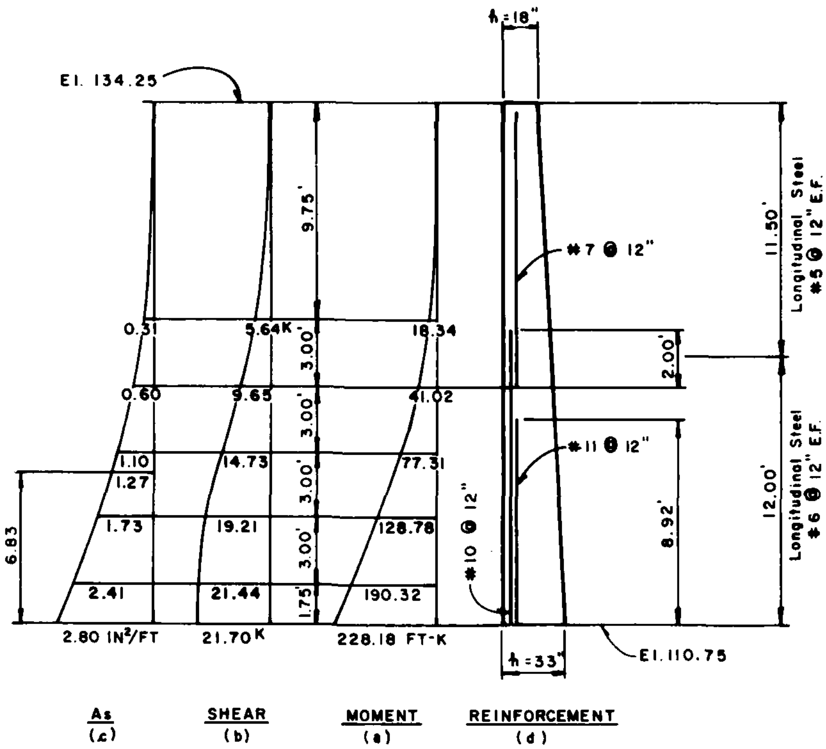


Figure 11. Moment, shear, and required reinforcement for stem

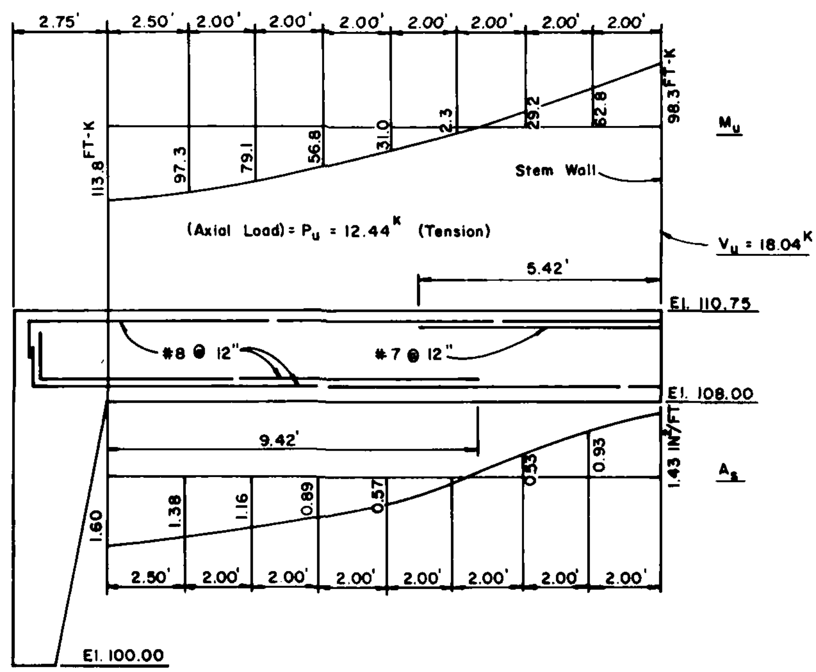


Figure 12. Moments and required reinforcement for heel

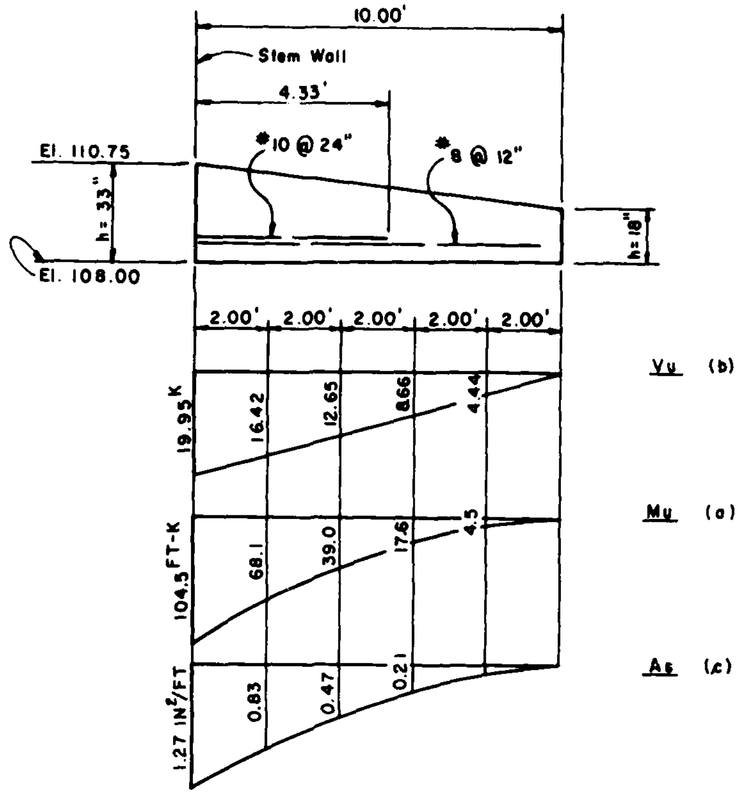


Figure 13. Moment, shear, and required reinforcement for toe

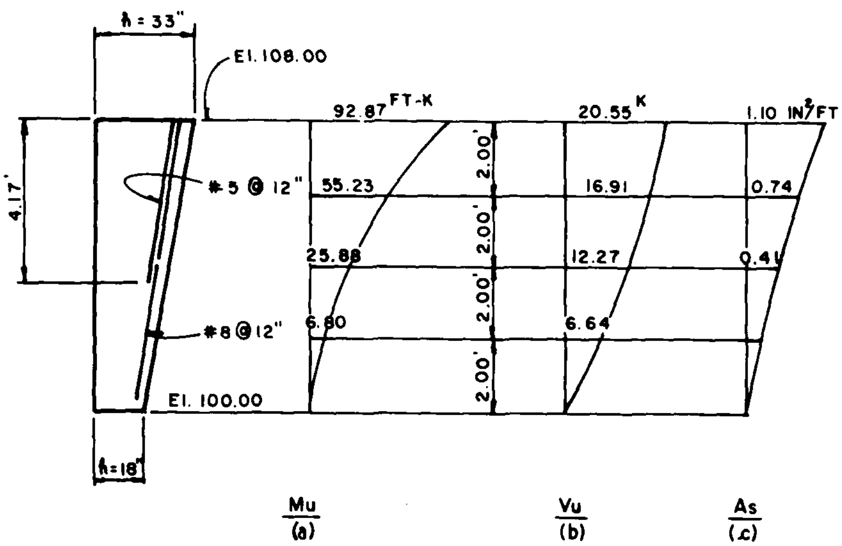


Figure 14. Moment, shear, and required reinforcement for key

into Equation 27 and solving for A_s, the answer obtained is $A_s = 2.80\ \text{in.}^2$.

76. Alternating No. 10 and No. 11 bars on 6-in. centers gives $A_s = 1.27 + 1.56 = 2.83\ \text{in.}^2/\text{ft}$. Check $\rho = 2.83/(12 \times 29) = 0.0081 < 0.25\ \rho_b$.

77. A_s for other points along the stem are determined in the same manner.

78. It can be seen from Figure 11c that the No. 11 bars are not required at a distance of 6.83 ft above the base of the stem. According to ACI 318-77, paragraph 12.11.3, reinforcement shall extend a distance of 12 bar diameters or the effective depth of the member (whichever is greater) past the point where it is no longer required. Twelve bar diameters for No. 11 bar is 16.92 in. Effective depth at 6.83 ft above base is approximately 25 in., which is greater than 16.92 in. Thus, the No. 11 bars can be discontinued at a point that is a distance of 8 ft 11 in. above the base. A check should be made in accordance with ACI 318-77, paragraph 12.11.5.1, to make certain that shear at the cut-off point does not exceed two-thirds of the permitted shear, as follows:

$$V_u\ (\text{8 ft 11 in. above base}) = 13{,}000\ \text{lb (Figure 11b)}$$

$$d = 14\ \text{in.} + \left(\frac{14.58}{23.5}\right) 15\ \text{in.} = 23.31\ \text{in.}$$

$$b = 12\ \text{in.},\quad V_c = 2\sqrt{f'_c} = 110\ \text{psi}$$

$$\phi V_c = 0.85 \times 110 \times 12 \times 23.31 = 26{,}150\ \text{lb}$$

$$2/3(\phi V_c) = 17{,}430\ \text{lb} > 13{,}000\ \text{lb}$$

79. It can be seen in Figure 11d that No. 7 bars at 12 in. are used for the top portion of the stem. From Figure 11c, No. 10 bars are no longer required at a point 10.75 ft above the base. Determine the distance above this point that the No. 10 bars must extend for proper lap with No. 7 bars, according to ACI 318-77, as follows:

a. Basic Development Length $= \dfrac{0.04\ A_b f_y}{\sqrt{f'_c}}$ (paragraph 12.2.2)

$$= \frac{0.04 \times 0.6 \times 40{,}000}{\sqrt{3000}} = 17.53 \text{ in.}$$

b. Modification Factor = $0.8\ \ell_d$ (paragraph 12.2.4)

$$= 0.8 \times 17.53 = 14.02 \text{ in.}$$

c. Lap Length = $1.7\ \ell_d$ (paragraphs 12.16.1 and 12.16.2)

$$= 1.7 \times 14.02 = 23.83 \text{ in. (24 in.)}$$

80. The shear should be checked at a distance $d = 2.29$ ft above the base of the stem as follows:

$$V_c = 2\sqrt{f'_c}\ bd \text{ (ACI 318-77, Equation 11-3)}$$

$$V_c = 2\sqrt{3000} \times 12 \times 2.29 \times 12 = 36{,}120 \text{ lb}$$

$$\phi V_c = 0.85 \times 36{,}120 = 30{,}700 \text{ lb}$$

$$V_u = 21{,}210 \text{ lb (Figure 11b)}$$

$$\phi V_c > V_u$$

Heel design

81. The moment diagram for the heel is shown in Figure 12. In addition to moment, the heel is subjected to a tensile load of 12.44 kips resulting from lateral loads on the key. The heel should be designed in accordance with the requirements of paragraph 43d.

82. The reinforcement in the top face should be designed for the moment existing at the point where the reinforcement in the back face of the stem intersects the base slab; as follows:

$$b = 12 \text{ in.},\quad h = 33 \text{ in.},\quad d = 29 \text{ in.}$$

$$M_u \text{ (at face of stem)} = 98.30 \text{ ft-K}$$

V_u (at fact of stem) = 18.04 K

M_u (at design point) = $98.30 + 18.04\left(\frac{4}{12}\right) = 104.30$ ft-K

Solve Equation 11 for

$$\rho = \frac{0.85\ f'_c k_u bd - P_n}{f_y bd} \tag{28}$$

and substitute into Equation 13 to solve for

$$k_u = 1 - \sqrt{1 - \frac{P_n e'}{0.425\ f'_c bd^2}} \tag{29}$$

$$P_n = \frac{P_u}{\phi} = \frac{-12.44}{0.9} = -13.83 \text{ K}$$

$$e' = \frac{M_u}{P_u} + \left(d - \frac{h}{2}\right) = \frac{12 \times 104.3}{-12.44} + \left(29 - \frac{33}{2}\right) = -88.10 \text{ in.}$$

$$k_u = 1 - \sqrt{1 - \frac{-13.83 \times -88.1}{0.425 \times 3 \times 12(29)^2}} = 0.0485$$

Substituting $k_u = 0.0485$ into Equation 28

$$\rho = \frac{0.85 \times 3 \times 0.0485 \times 12 \times 29 + 13.83}{40 \times 12 \times 29} = 0.0041$$

$$A_s = \rho bd = 0.0041 \times 12 \times 29 = 1.43 \text{ in.}^2$$

83. No. 8 bars alternating with No. 7 bars on 6-in. centers giving $A_s = 1.39$ in.^{2} will be used, furnishing 97.20 percent of required area.

84. The required reinforcement for other points along the heel can be determined in the same manner, and are shown in Figure 12.

85. Check shear at the face of the stem as follows:

$$V_c = 2\left(1 - \frac{N_u}{500\ A_g}\right)\sqrt{f'_c}\ bd \text{ (ACI 318-77, Equation 11-9)}$$

$$V_c = 2\left(1 - \frac{12,440}{500 \times 396}\right)\sqrt{3000}\ (12 \times 29) = 35,730 \text{ lb}$$

$$\phi V_c = 0.85 \times 35,730 = 30,370 \text{ lb}$$

$$V_u = 18,040 \text{ lb} < \phi V_c$$

86. The No. 7 bar in the top face is no longer required at a point 3 ft from the stem. The effective depth is 29 in., and 12 bar diameters equals 9 in. The cutoff point is at 29 in. + 36 in. = 65 in. = 5.42 ft from the stem. Check the shear at the cutoff in accordance with ACI 318-77, paragraph 12.11.5.1 as follows:

$$V_u \text{ (at cutoff)} = 15,489 \text{ lb}$$

$$\phi V_c \text{ (computed above)} = 30,370 \text{ lb}$$

$$2/3\ \phi V_c = 20,250 \text{ lb} > 15,489 \text{ lb}$$

87. The cutoff point for a No. 8 bar in the bottom face is determined in the same manner.

Toe design

88. The factored moments and shears for the toe are shown in Figure 13a and b. The required reinforcement at the face of the stem can be determined from Equation 27 by substituting

$M_u = 104.5 \times 12 = 1254$ in.-K

$\phi = 0.9$

$b = 12$ in.

$d = 33 - 5 = 28$ in.

$f'_c = 3$ ksi

$f_y = 40$ ksi

into Equation 27, and solving for $A_s = 1.27$ in.^{2}.

88. Use No. 10 bars on 24-in. centers with No. 8 bars on 12-in. centers giving $A_s = 1.42$ in.^{2}. Check $\rho = 1.42/(12 \times 28) = 0.0042 < 0.25\ \rho_b$.

89. The shear should be checked at distance 2.07 ft from the face of the stem, where the effective depth is 2.07 ft or 24.89 in., as follows:

$$V_u = 16{,}287 \text{ lb}, \quad b = 12 \text{ in.}, \quad d = 24.89 \text{ in.}, \quad \rho = 0.00465$$

$$M_u = 66.94 \times 12 = 803 \text{ in.-K}$$

$$V_c = \left(1.9\sqrt{f'_c} + \frac{2500\ \rho V_u d}{M_u}\right) bd \quad \text{(ACI 318-77, Equation 11-6)}$$

$$V_c = \left(1.9\sqrt{3000} + \frac{2500 \times 0.00465 \times 16.287 \times 24.89}{803}\right)(12 \times 24.89)$$

$$= 32{,}836 \text{ lb}$$

$$\phi V_c = 0.85 \times 32{,}836 = 27{,}910 \text{ lb} > V_u$$

90. The No. 7 bar is not required for flexure at a point 2.25 ft from the stem, where the effective depth is 24.89 in. Twelve bar diameters = 9.0 in. The cutoff point then is 24.89 in. + 27 in. = 51.89 in. = 4.32 ft (use: 4.33 ft). Check the shear at the cutoff point in accordance with ACI 318-77, paragraph 12.11.5.1 as follows:

$$V_u = 12{,}000 \text{ lb}, \quad b = 12 \text{ in.}, \quad d = 21.50 \text{ in.}$$

$$V_c = 2\sqrt{f'_c}\ bd \quad \text{(ACI 318-77, Equation 11-3)}$$

$$= 2\sqrt{3000}\ (12 \times 21.50) = 28{,}262 \text{ lb}$$

$$2/3(\phi V_c) = 2/3 \times 0.85 \times 28{,}262 = 16{,}015 \text{ lb} > 12{,}000$$

Key design

91. Figure 14a and b show the factored moment and shear for the key. The required reinforcement can be determined from Equation 27 and is shown in Figure 14c. It can also be determined that the critical shear at the bottom of the base is less than ϕV_c specified in ACI 318-77.

Longitudinal steel

92. Longitudinal bars are used to space the flexure bars and to provide for shrinkage and temperature stresses. According to paragraph 36, the area of shrinkage and temperature reinforcement in each direction should not be less than 0.1 percent of the gross cross-sectional area. At the base of the stem, the required A_s would be:

$$0.001 \times 12 \times 33 = 0.396 \text{ in.}^2$$

The steel used is, in part, a matter of judgment. The following steel for the stem is chosen:

Longitudinal, upper 11.50 ft, No. 5 at 12 in., each face

$$A_s = 0.31 \text{ in.}^2/\text{ft}$$

below 11.50 ft, No. 6 at 12 in., each face

$$A_s = 0.44 \text{ in.}^2/\text{ft}$$

Vertical at land face, No. 6 at 12 in.

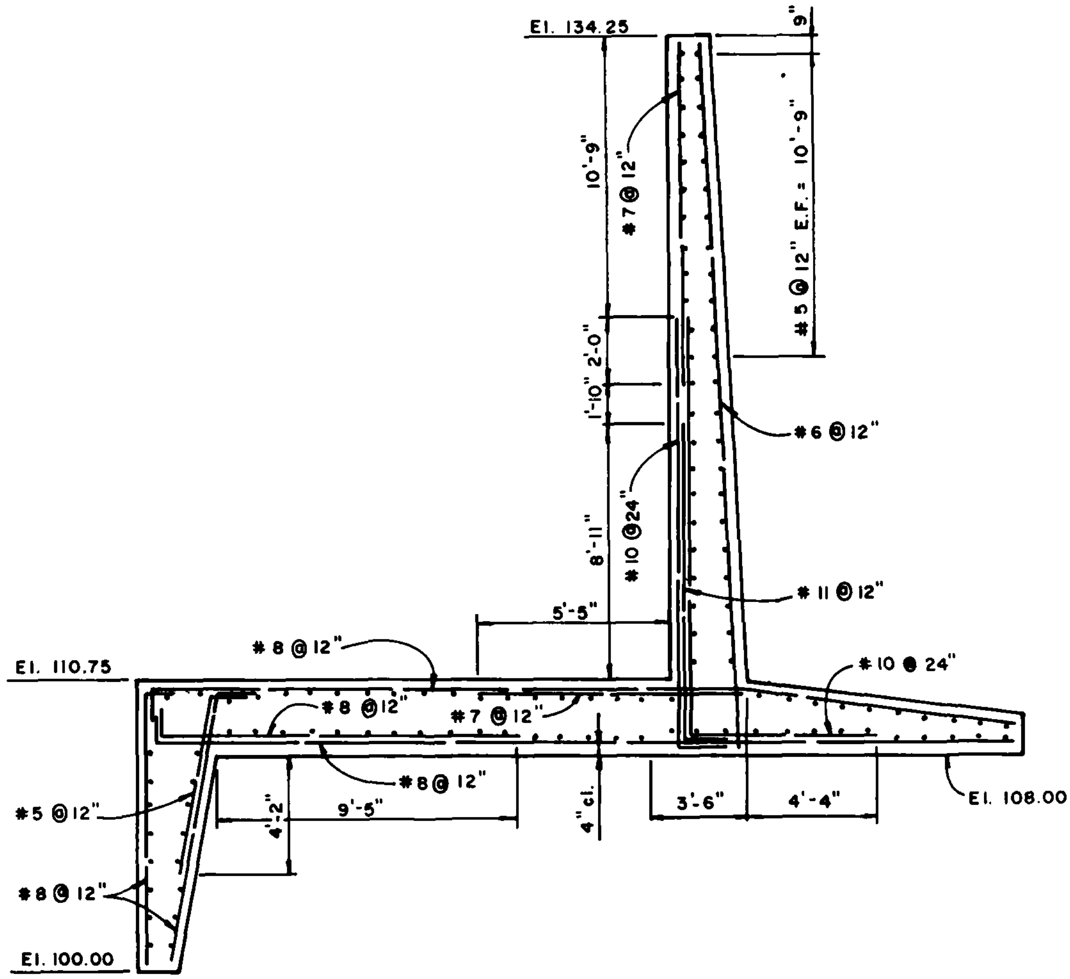
$$A_s = 0.44 \text{ in.}^2/\text{ft}$$

Longitudinal reinforcement in all other members (heel, toe, and key) are all No. 6 at 12 in.

$$A_s = 0.44 \text{ in.}^2/\text{ft}$$

Reinforcement summary

93. The reinforcement summary is given in Figure 15.



NOTES:

1. All longitudinal reinforcement to be #6 @ 12" E.F. except as noted.
2. Concrete cover for reinforcement to be 3 inches except as noted.
3. All hooks shown to be ACI standard hooks.

Figure 15. Reinforcement summary

REFERENCES

Al-Hussaini, M. M., and Townsend, F. C. 1975 (Dec). "Investigation of K_o Testing in Cohesionless Soils," Technical Report S-75-16, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

American Concrete Institute. 1977. "Building Code Requirements for Reinforced Concrete," ACI Manual of Concrete Practice, ACI 318-77, Detroit, Mich.

Atimay, E., and Ferguson, P. 1973 (Sep). "Early Chloride Corrosion of Reinforced Concrete - A Test Report," ACI Journal, Detroit, Mich.

Beeby, A. W. 1978. "Concrete in the Oceans - Cracking and Corrosion," Technical Report No. 1, Cement and Concrete Association, Wexham Springs, England.

Bowles, J. E. 1968. Foundation Analysis and Design, McGraw-Hill, New York.

British Standards Institution. 1972. "The Structural Use of Concrete; Part 1: Design Materials and Workmanship," CP 110.

Broms, B. B. 1965a (Oct). "Crack Width and Crack Spacing in Reinforced Concrete Members," ACI Journal, Proceedings, Vol 62, No. 10, pp 1237-1256.

__________. 1965b (Sep). "Stress Distribution in Reinforced Concrete Members with Tension Cracks," ACI Journal, Proceedings, Vol 62, No. 9, pp 1095-1108.

__________. 1965c (Jan). "Techniques for Investigation of Internal Cracks in Reinforced Concrete Members," ACI Journal, Proceedings, Vol 62, No. 1, pp 35-44.

Clark, A. P. 1956 (Apr). "Cracking in Reinforced Concrete Flexural Members," ACI Journal, Proceedings, Vol 52, No. 8, pp 851-862.

Comite European-International du Beton. 1978 (Apr). "Code Modele CEB-FIP pour les Structures en Beton," Bulletin d'Information No. 1240/125-F.

Concrete Reinforcing Steel Institute. 1978. CRSI Handbook, 3rd ed.

Federation Internationale de la Precontrainte (FIP). 1974. "Recommendations for the Design and Construction of Concrete Sea Structures," 2nd ed.

Gergely, P., and Lutz, L. A. 1968. "Maximum Crack Width in Reinforced Concrete Flexural Members," Causes, Mechanism, and Control of Cracking in Concrete, SP-20, American Concrete Institute, Detroit, Mich., pp 87-117.

Goto, Y. 1971 (Apr). "Cracks Formed in Concrete Around Deformed Tension Bars," ACI Journal, Proceedings, Vol 68, No. 4, Detroit, Mich.

Hognestad, E. 1962 (Jan). "High Strength Bars as Concrete Reinforcement: Part 2 - Control of Cracking," Journal, PCA Research and Development Laboratories, Vol 4, No. 1, pp 46-62.

Huntington, W. C. 1957. Earth Pressures and Retaining Walls, John Wiley and Sons, New York.

Husain, S. I., and Ferguson, P. M. 1968 (Jun). "Flexural Crack Width at the Bars in Reinforced Concrete Beams," Research Report 102-IF, Center for Highway Research, The University of Texas, Austin, Tex.

Illston, J. M., and Stevens, R. F. 1972 (Dec). "Long-Term Cracking in Reinforced Concrete Beams," Proceedings, the Institution of Civil Engineers, Part II, Research and Theory, Vol 53.

Kaar, P. H., and Hognestad, E. 1965 (Jan). "High Strength Bars as Concrete Reinforcement: Part 7 - Control of Cracking in T-Beam Flanges," Journal, PCA Research and Development Laboratories, Vol 7, No. 1, pp 42-53.

Kaar, P. H., and Mattock, A. H. 1963 (Jan). "High Strength Bars as Concrete Reinforcement; Part 4, Control of Cracking," Journal, PCA Research and Development Laboratories, Vol 5, No. 2, pp 15-38.

Kennedy, T. B. 1955 (Jul). "Tensile Crack Exposure Tests; CWI Item No. 026, Tensile Crack Exposure Tests for Reinforced Concrete Beams," Technical Memorandum No. 6-412, Report 1, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Klein, F., Hoffman, E. S., and Rice, P. 1979 (18-23 Mar). "Application ACI Committee 350 of Strength Design Methods to Sanitary Structures," Paper presented at ACI Manual Convention at Milwaukee, Wis.

Liu, T. C. 1980 (Jul). "Strength Design of Reinforced Concrete Hydraulic Structures; Preliminary Strength Design Criteria," Technical Report SL-80-4, Report 1, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Martin, H., and Schiessl, P. 1969a. "The Influence of Cracks on the Corrosion of Steel in Concrete," Preliminary Report of RILEM International Symposium on the Durability of Concrete, Vol II, Prague, Czechoslovakia.

__________. 1969b. "The Influence of Time and Environmental Conditions on the Corrosion of Deformed Bars in Cracked Concrete," Preliminary Report of RILEM International Symposium on the Durability of Concrete, Vol II, Prague, Czechoslovakia.

Okamoto, S. 1956. "Bearing Capacity of Sandy Soil and Lateral Earth Pressure During Earthquakes," Proceedings, World Conference on Earthquake Engineering, Berkeley, Calif.

O'Neil, E. F. 1980 (Jan). "Tensile Crack Exposure Tests; Laboratory Evaluation of Series 'A' Beams with Results from 1951 to 1975," Technical Memorandum No. 6-412, Report 3, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Quigley, D. W., and Duncan, J. M. 1978 (Sep). "Earth Pressure on Conduits and Retaining Walls," Report No. U2B/GT/78-06, Department of Civil Engineering, University of California, Berkeley, Calif.

Rangan, B. V., and Rajasekar, H. L. 1977 (30 Aug). "Serviceability and Strength Design Provisions for Concrete Water-Retaining Structures," Paper presented at Concrete Symposium, Brisbane, Australia.

Reis, E. E., et al. 1965. "Causes and Control of Cracking in Concrete Reinforced with High-Strength Steel Bars - A Review of Research," Bulletin No. 479, Engineering Experiment Station, University of Illinois, Urbana, Ill.

RILEM. 1957. "Symposium on Bond and Crack Formation in Reinforced Concrete; Vols I and II," Paris, France.

Roshore, E. C. 1964 (Nov). "Tensile Crack Exposure Tests; Results of Tests of Reinforced Concrete Beams, 1955-1963," Technical Memorandum No. 6-412, Report 2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Rusch, H., and Rehm, G. 1963, 1964. "Versuche Mit Betonformstahlen," Deutscher Ausschuss fur Stahlbeton, Bulletins, No. 140, Part 1, 1963; No. 160, Part 2, 1963; and No. 165, Part 3, 1964.

Seed, H. B., and Whitman, R. V. 1970. "Design of Earth Retaining Structures for Dynamic Loads," Proceedings, 1970 ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures.

Spangler, M. G. 1961. Soil Engineering, 2nd ed., International Textbook Company.

Teng, W. C. 1962. Foundation Design, Prentice-Hall, Inc., New Jersey.

Terzaghi, K., and Peck, R. B. 1948. Soil Mechanics in Engineering Practice, John Wiley & Sons, New York.

Tremper, B. 1947 (Jun). "The Corrosion of Reinforcing Steel in Cracked Concrete," ACI Journal, Detroit, Mich.

U. S. Army Coastal Engineering Research Center, CE. 1973. Shore Protection Manual, Vol II, Ft. Belvoir, Va.

U. S. Army, Office, Chief of Engineers. 1948 (Jan). "Wall Design Flood Walls," Engineer Manual EM 1110-2-2501, Washington, D. C.

__________. 1958 (Sep). "Gravity Dam Design," Engineer Manual EM 1110-2-2200, Washington, D. C.

__________. 1961 (29 May). "Engineering and Design; Retaining Walls," Engineer Manual EM 1110-2-2502, Washington, D. C.

__________. 1963. "Working Stresses for Structural Design," Engineer Manual 1110-1-2101, Washington, D. C.

__________. 1971 (May). "Details of Reinforcement - Hydraulic Structures," Engineer Manual EM 1110-2-2103, Washington, D. C.

__________. 1977 (Apr). "Earthquake Design and Analysis for Corps of Engineers Dams," Engineer Regulation ER 1110-2-1806, Washington, D. C.

U. S. Army, Office, Chief of Engineers. 1979 (Mar). "Criteria Specifications for Computer Program TWDA, T-Wall Design/Analysis for Analysis and/or Design of Inverted-T Retaining Walls and Flood Walls," LMVD CASD Committee and the OCE CASE Project Task Group on T-Wall Design, Washington, D. C.

__________. 1980 (Mar). "User Reference Manual for Computer Program TWDA, T-Wall Design/Analysis for Analysis and/or Design of Inverted-T Retaining Walls and Flood Walls" (Working Draft), LMVD CASD Committee and the OCE CASE Project, Task Group on T-Wall Design, Washington, D. C.

APPENDIX A: COMMENTARY

Introduction

1. This appendix discusses some of the considerations and background information used in developing the strength design criteria for inverted T-walls used as reinforced concrete retaining walls or floodwalls presented in Part II.

General Design Criteria

2. The design provisions of ACI 318-77 are generally applicable for reinforced concrete retaining walls and floodwalls. However, because of the uncertainties of the loading conditions and serviceability requirements of the retaining walls and floodwalls, some design criteria of ACI 318-77 need to be modified. The specific design criteria that are different from those of ACI 318-77 are given in Part II.

Loads and Forces

3. The following discussion describes the various forces that may act on a wall. General procedures for computing their magnitudes are also included. In considering walls in which the cross section remains constant for a considerable distance, the computation is based on a linear foot of wall.

Dead load (d)

4. The resultant of the weight of the wall acts through the center of gravity of the portion above the horizontal section considered. It is usually simpler to divide a cross section into triangles and rectangles, whose areas and centroids can be determined easily, and to deal with the corresponding forces separately rather than to combine these into a single force.

Vertical earth pressure (H_w)

5. The vertical earth pressure is equal to the depth of the

plane below the ground surface multiplied by the unit weight of the soil.

6. The method for determining the unit weight of soil for use in computation of earth pressure depends primarily upon the size and economic importance of the wall. For small walls or for projects involving relatively small expenditures, the cost of laboratory and field tests is usually not justified. However, a simple exploratory program should be carried out to determine the general character of the foundation materials and to permit classification of the soils in the borrow areas that will provide the backfill. The probable level of the water table after construction should also be estimated on the basis of local conditions. With this information, sufficiently reliable values of the appropriate unit weights can usually be taken from Table A1.

7. In connection with high walls, or important projects involving large expenditures, a thorough study of available backfill materials is advisable. Laboratory field tests should be made to determine the saturated, drained, and dry weights of all materials that may be used, and the design should be based on the test values.

Lateral earth pressure (H_p)

8. Active earth pressure. For all practical purposes, Coulomb's theory can be used to predict the active earth pressures. The equation for computing active pressure coefficient given in EM 1110-2-2502 is convenient for homogeneous backfills and simple geometric installations. For a more complicated multiple-layer soil system, the incremental trial wedge method presented in U. S. Army, Office, Chief of Engineers (1979)* may be used.

9. For unusual wall conditions, the finite element method can be a valuable tool for design. At the present time, the finite element method is the only practical means by which quantitative analyses can be made of soil-structure interaction problems.

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* The references cited in this appendix are listed in the references at the end of the main text.

10. At-rest earth pressure. The at-rest earth pressures specified in EM 1110-2-2502 are estimated from empirical coefficients related to soil type. The Jaky's equation and field or laboratory tests can also yield a suitable value for at-rest earth pressure coefficients. More detailed discussions of at-rest earth pressure can be found in U. S. Army, Office, Chief of Engineers (1979) and Al-Hussaini and Townsend (1975).

11. Passive earth pressure. The accuracy of predictions of passive earth pressures which can be relied upon for resisting the movement of retaining walls and floodwalls is considerably less than the accuracy of predictions of active pressures. Because of the nonlinear relationship between passive resistance and wall movement, there remains much uncertainty as to whether the desired resistance can in fact be achieved within the wall movement allowed. Therefore, passive earth pressures are often disregarded when other more positive means of resistance are available (Quigley and Duncan 1978).

12. Location of resultant lateral pressure. The point of application of the earth pressure resultant for a cohesionless material is controlled largely by the amount of wall movement. The exact location is generally indeterminate. However, it is sufficiently accurate to use the locations of the earth pressure resultant specified in EM 1110-2-2502.

Vertical water pressure (F_w)

13. The vertical water pressure is equal to the depth of the water multiplied by 62.5 lb/ft^3.

Lateral water and seepage pressure (F_p)

14. For a wall with a permeable backfill (assuming no seepage under the wall) and no drainage system provided, the pressure on the wall is due to the buoyant weight of the earth plus the full hydrostatic pressure of the water.

15. When seepage is taking place in a floodwall or a retaining wall provided with a drainage system, the pore pressure is reduced by friction between the soil grains and the water. The most effective way to determine the pore pressure is by means of the creep method or the flow net method, as described in EM 1110-2-2501.

16. Water may accumulate in cracks that form in a backfill of cohesive soil. The hydrostatic pressure due to this water may have significant effect on the lateral pressure against a wall. Therefore, full hydrostatic pressure should be used in the design, if such water is adjacent to the back of a wall.

Uplift (F_u)

17. The common procedure for computing uplift pressure on walls resting on soil is the creep method, which is much simpler than using a flow net.

18. When the resultant force falls outside the kern and the assumed failure plane is along the interface between the base of the wall and the soil, a crack at soil-structure interface will form for the portion of the foundation not in compression, and therefore, no creep loss should be assumed for this portion.

Wind load (W)

19. EM 1110-2-2502 specifies that a horizontal loading of 30 psf should be used as wind load for retaining walls. For floodwall design, EM 1110-2-2501 specifies that, in localities subject to hurricanes or cyclones, a 50-psf wind load should be used; and for all other localities, a wind load of 20 psf should be used.

Wave action (p)

20. Wave pressures due to breaking and nonbreaking waves differ widely. The first step in the evaluation of wave force on floodwalls is to determine if the wall will be subjected to forces from nonbreaking waves, breaking waves, or broken waves. The determination of wave pressure on vertical walls is explained in Shore Protection Manual,

Vol II (U. S. Army Coastal Engineering Research Center 1973).

Surcharge loads (S_L)

21. The usual procedure in providing for uniform surcharge loads is to apply to the ground surface an imaginary equivalent surcharge of earth with the same unit weight as the backfill and with a height sufficient to produce the same intensity of vertical pressure on the ground surface as is applied by the actual load. However, the direct methods (e.g., Boussinesq's equations (Terzaghi and Peck 1948, Spangler 1961) or theory of elasticity (Bowles 1968)) are generally more suitable for concentrated loads.

Earthquake loads (E)

22. The seismic coefficient method is generally used for computing earthquake loads. The horizontal initial force can be obtained by multiplying a seismic coefficient, which represents ~~the~~ ratio of an assumed acceleration of structure to the acceleration of gravity, and the weight of the structures. The dynamic water loads on vertical or near-vertical surfaces of the structure are usually determined by the Westergaard formula (U. S. Army, Office, Chief of Engineers 1958) using the selected seismic coefficient. The dynamic horizontal earth pressure magnitude and resulting force can be approximated by the Mononobe-Okabe method (Okamoto 1956, Seed and Whitman 1970).

23. In addition to a seismic coefficient analysis, the seismic design of a wall that is used as part of a dam and when failure of the wall could result in loss of life or extensive property damage, should include dynamic methods as described in ER 1110-2-1806.

Other structural effect (T)

24. The effect of differential settlement, creep, shrinkage, and temperature changes should be considered in T-wall design. In the design, the most probable values rather than the upper bound values of the variables should be used.

Strength Requirements

Required strength

25. The required strength U is expressed in terms of factored loads, which are the loads discussed in paragraphs 10-27 (main text), multiplied by appropriate load factors. The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. In the T-wall design, the load factor for dead load and the effects of differential settlement, creep, shrinkage, and temperature change is 1.5. All other loads have a load factor of 1.9. The basis for using these load factors is given in Liu (1980).

26. Paragraph 28 (main text) gives load factors for specific combinations of loads. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered. Consideration must be given to various combinations of loading to determine the most critical design conditions. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load in the heel of the floodwall.

27. The load combinations with 0.9D are included in Equations 2 and 3 for the case where a higher dead load reduces the effects of other loads.

28. The reduction factor of 0.75 used in Equations 4-6 is consistent with the provision of increasing the allowable stresses by 33-1/3 percent for Group II loadings (U. S. Army, Office, Chief of Engineers 1963) for the working-stress design.

Base reaction

29. Paragraph 29 (main text) requires that the wall footings be proportioned to sustain the applied factored loads and induced reactions.

30. In general, the size of the T-wall base on soil is established or the basis of service loads in whatever combination will

govern the design and without applying any load factors. The extreme soil pressure obtained from this loading must be within the permissible values, and the overturning and sliding criteria should also be satisfied.

31. To proportion a T-wall base for strength, the base reaction due to the applied "factored" loading must be determined.

32. In the case of a T-wall, load factors will cause eccentricities and reactions that are different from those obtained by unfactored loads. However, it is important to note that the base reaction is only a calculated reaction to the factored loading used to produce, in the base, the same required strength conditions regarding flexure and shear as in the stem.

33. It will be shown in Appendix B that in the typical retaining wall and floodwall designs, the factored loads will only cause slight relocation of the resulting eccentricities from those obtained under service load conditions, and the overall load factors for moment and shear at critical sections due to the applied factored loads and induced reactions are in the order of 1.9 for the loading combination given in Equation 1 in paragraph 28 (main text).

Design strength for reinforcement

34. The basis for allowing the use of Grade 60 reinforcement for T-walls is as follows:

a. A recent survey indicated that Grade 60 reinforcement is currently being specified for use in the vast majority of reinforced concrete structures, and the ASTM is now considering a proposal to delete bar size No. 7 and larger in Grade 40.

b. Grade 40 reinforcement has been specified for T-walls. When Grade 40 reinforcement is not immediately available, it is not unusual to substitute standard Grade 60 reinforcement for Grade 40 reinforcement on a bar-for-bar basis. However, a substitution without a redesign of the splices and anchorages is not a sound practice. A simple substitution can change the mode of failure of a structure under overload from ductile to brittle by increasing the flexural strength safety factor without proportional changes in the safety factors for shear or development of reinforcement.

c. All existing design codes and specifications, such as ACI Building Code (ACI 318-77), American Association of State Highway and Transportation Officials (AASHTO) design specifications, American Railway Engineering Association (AREA) design specifications, Uniform Building Code (UBC), and ACI Committee 350 recommendations on "Application of Strength Design Methods to Sanitary Structures," allow the use of Grade 60 reinforcement. In general, for working stress design, the tensile stress of Grade 60 reinforcement is limited to 24,000 psi, and for strength design, a check of the crack width at service loads is required for design yield strength f_y exceeding 40,000 psi.

d. Grade 60 reinforcement has replaced Grade 40 reinforcement as the industry standard, and design aids based on Grade 40 reinforcement are now becoming obsolete. It is noted that the CRSI Handbook's (Concrete Reinforcing Steel Institute 1978) design aid tables utilize the Grade 60 reinforcement only.

e. The use of Grade 60 reinforcement will generally result in a more economical design without compromising serviceability and safety requirements. For a 1 percent steel ratio, approximately 8 percent cost saving can be realized by using Grade 60 reinforcement (f_y = 48,000 psi) instead of Grade 40.

Serviceability Requirement

Distribution of flexural reinforcement

35. The subject of crack control and its relation to corrosion is presented in Appendix C in that the general background information on crack mechanism is reviewed, an appraisal of the available field exposure test data relevant to crack widths and corrosion is made, and the existing design methods of controlling crack width are evaluated. Based on the information gathered for Appexdix C, the following conclusions were made:

a. Cracks in the width covered in exposure tests (up to 0.06 in.) will be likely to induce corrosion where bars intersect them, but the amount of corrosion occurring at the cracks over the design life of the structure will not be significantly influenced by the width of the cracks.

b. Most codes and regulations require either a direct or indirect check on crack width, but these checks, though intended to control possible corrosion, are based on no sound foundation of data relating crack width to corrosion. Nor is there any general agreement as to whether crack widths should be checked or how this check should be carried out.

c. It appears that a design check on the crack widths, either at points directly over main bars or on the surface, is irrelevant from the point of view of corrosion protection.

d. With the specified thick concrete cover, low steel stress, and small diameter bars and spacings for reinforced concrete hydraulic structures, there is no reason to expect extensive corrosion problems during their design life, regardless of cracking.

36. Since the tension zone is in the backfill face of a retaining wall where cracks are not objectionable for appearance and where bituminous coatings can be employed if salt water or other corrosion-aggressive solution is expected, the arbitrary maximum bar spacing requirements specified in paragraph 32 (main text) are considered adequate.

Control of deflections

37. The provisions of paragraphs 34 and 35 (main text) are concerned only with the deflections or deformations that may occur at service load levels.

38. The walls designed by the working-stress method have low concrete and steel stresses and have served their intended functions with very limited deflections. Paragraph 34 (main text) specifies that deflections at service loads need not be checked when the reinforcement ratio does not exceed $0.25 \rho_b$.

39. The use of large steel ratios in the strength-design method will generally result in more slender sections than those designed by the working-stress design method. Because the deflection of slender members may in some cases exceed desirable limits, paragraph 35 (main text) specifies that deflection should be checked when the reinforcement ratio exceeds $0.25 \rho_b$.

Shrinkage and
temperature reinforcement

42. Shrinkage and temperature reinforcement is required at the exposed faces to prevent excessive cracking. The amounts specified are empirical but have been used satisfactorily for T-walls.

43. In the backfill face of a retaining wall, temperature and shrinkage exposure is ordinarily less severe than for exposed face. Appearance, watertightness, etc., are ordinarily not design requirements. Longitudinal bars at spacings not to exceed approximately 4 ft are generally required as practical bar supports and spacers to hold the flexural reinforcement in place during construction.

Concrete cover for reinforcement

44. Concrete cover as protection of reinforcement against weather and other effects is measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. The concrete covers specified in paragraph 37 (main text) are consistent with the requirements for thick concrete sections (3 ft or more) not exposed to action of water, alkali, other destructive agents, or severe impact or abrasion given in EM 1110-2-2103 (U. S. Army, Office, Chief of Engineers 1971).

Details of reinforcement

45. Good structural details are vital to the satisfactory performance of reinforced concrete structures. The standard practice for reinforcement details for concrete hydraulic structures is given in EM 1110-2-2103.

Flexure and Axial Load

Maximum tension reinforcement

46. The maximum amount of tension reinforcement in flexural members is limited to ensure a level of ductile behavior. For T-walls, the maximum tension reinforcement is limited to $0.25\ \rho_b$. The derivation of this limitation is given in Liu (1980).

47. For unusual situations when a larger amount of reinforcement

is necessary because of physical constraints on member dimensions due to functional or other requirements, paragraph 40 (main text) limits the reinforcement ratio to $0.50\ \rho_b$ provided that the deflection criteria specified in paragraph 35 (main text) are met.

Minimum reinforcement of flexural members

48. This provision applies to members that for functional or other reasons are much larger in cross section than required by strength consideration. The computed moment strength as a reinforced concrete section with very little tensile reinforcement becomes less than that of the corresponding plain concrete section computed from its modulus of rupture. Failure in such a case could be quite sudden. A minimum percentage of reinforcement should be provided to prevent such a mode of failure.

Combined flexure and axial load

49. Design assumptions and general requirements.

a. The allowable strain ε_M at extreme concrete compression fiber is limited to 0.0015. This value is smaller than 0.0030 assumed in the ACI 318-77. The bases for selecting this lower value are as follows:

(1) It can be shown that the use of ε_M of 0.0015 will yield designs that are comparable to those designed by the working-stress method with allowable concrete compressive stress of $0.45\ f_c'$. It is expected that the use of ε_M of 0.0015 will result in designs satisfying both the strength and serviceability requirements of T-walls.

(2) Concrete strain at maximum stress has been observed in tests of various kinds to vary from 0.0015 to 0.0020. The use of ε_M of 0.0015 recognizes the in elastic stress distribution of concrete at high stress.

(3) The use of higher ε_M value (say 0.003) will result in a thinner section for some combinations of moment and thrust. While the thinner section may satisfy the strength requirement, the serviceability requirement cannot be assured.

b. Balanced strain conditions for a cross section are defined as the conditions when the maximum strain at the extreme compression fiber just reaches 0.0015 simultaneously with the first yield strain in the tension reinforcement. The balanced load strength ϕP_b, and balanced moment strength ϕM_b should be computed in accordance with the appropriate equations given in paragraph 43 (main text).

c. The actual distribution of concrete compressive stress in a practical case is complex and usually not known explicitly. For practical design, paragraph 43 (main text) allows the use of a rectangular compressive stress distribution to replace the more exact concrete stress distributions. In the equivalent rectangular stress block, an average stress of $0.85 f'_c$ is used with a rectangle of depth $a = \beta_M c$.

d. The β_M of 0.55 for concrete with $f'_c \leq 4000$ psi and 0.05 less for each 1000 psi of f'_c in excess of 4000 is derived in Appendix D.

e. The eccentricity ratio e'/d is defined in Equation 7. It should be noted that P_u is positive for compression and negative for tension.

50. Flexural and compression capacity--tension reinforcement only.

a. The design axial load strength at zero eccentricity ϕP_o may be computed by:

$$\phi P_o = \phi[0.85 f'_c(A_g - \rho bd) + E_s \varepsilon_M \rho bd] \quad \text{(A1)}$$

For design, the axial load strength of compression members is limited to 70 percent of the design axial load strength at zero eccentricity. This percentage value is obtained by calibrating the strength design to the results of working-stress design with allowable concrete compressive stress of $0.35 f'_c$.

b. The e'_b/d at balanced condition can be derived as follows. For singly reinforced members subjected to combined flexure and compressive axial load, the equilibrium between external and internal forces and moments can be written as (see Figure A1)

$$\frac{P_u}{\phi} = 0.85 f'_c k_u bd - f_{su} \rho bd \quad \text{(A2)}$$

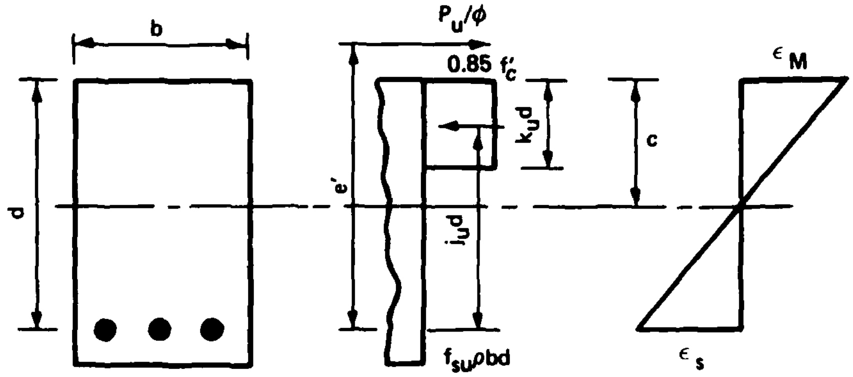

Figure A1. Stresses and strains in member subjected to combined flexure and axial load--tension reinforcement only

and

$$\frac{P_u e'}{\phi} = 0.85\ f'_c k_u bdj_u d \tag{A3}$$

Since

$$j_u = 1 - \frac{k_u}{2} \tag{A4}$$

Equation A3 can be rewritten as

$$\frac{P_u e'}{\phi} = 0.425\ f'_c \left(2k_u - k_u^2\right) bd^2 \tag{A5}$$

The stress-strain distributions at balanced condition are shown in Figure A2. From the strain diagram (Figure A2c), it can be shown that

$$\frac{\varepsilon_s + \varepsilon_1}{1 - k_b} = \frac{\varepsilon_M - \varepsilon_1}{k_b} \tag{A6}$$

and

$$\varepsilon_1 = \varepsilon_M (1 - \beta_M) \tag{A7}$$

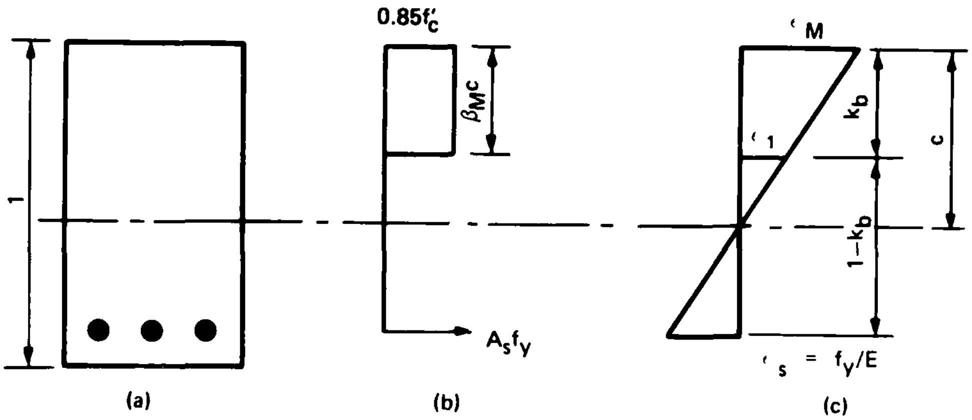

Figure A2. Stress and strain distributions at balanced condition

After substituting Equation A7 into Equation A6 and rearranging, the results obtained are

$$k_b = \frac{\beta_M E_s \varepsilon_M}{E_s \varepsilon_M + f_y} \tag{A8}$$

Since

$$e'_b = \frac{P_b e'}{P_b} \tag{A9}$$

e'_b can be obtained by substituting Equations A5 and A2 into Equation A9 with $k_u = k_b$, $f_{su} = f_u$ and $P_u = P_b$.

$$e'_b = \frac{0.425\ f'_c\left(2k_b - k_b^2\right)bd^2}{0.85\ f'_c k_b bd - f_y \rho bd} \tag{A10}$$

Therefore,

$$\frac{e'_b}{d} = \frac{2k_b - k_b^2}{2k_b - \dfrac{f_y \rho}{0.425\ f'_c}} \tag{A11}$$

c. For sections controlled by tension, the ϕP_n can be obtained from Equation A2 with $f_{su} = f_y$,

$$\phi P_n = \phi(0.85\ f'_c k_u bd - f_y \rho bd) \tag{A12}$$

The design moment strength ϕM_n can be expressed as

$$\begin{aligned} \phi M_n &= \phi P_n e \\ &= \phi P_n \left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] d \end{aligned} \tag{A13}$$

Therefore,

$$\phi M_n = \phi(0.85\ f'_c k_u bd - f_y \rho bd)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right] d \tag{A14}$$

Substituting Equation A2 with $f_{su} = f_y$ into Equation A5 gives

$$(0.85\ f'_c k_u bd - f_y \rho bd)e' = 0.425\ f'_c\left(2k_u - k_u^2\right)bd^2 \tag{A15}$$

Equation A15 can be reduced to

$$k_u^2 + 2\left(\frac{e'}{d} - 1\right)k_u - \frac{f_p \rho e'}{0.425\ f'_c d} = 0 \tag{A16}$$

d. For sections controlled by compression, the ϕP_n can be obtained from Equation A2.

$$\phi P_n = \phi(0.85\ f'_c k_u bd - f_{su} \rho bd) \tag{A17}$$

The ϕM_n can be obtained by multiplying Equation A17 by e , i.e.,

$$\phi M_n = \phi P_n e$$

$$\phi M_n = \phi(0.85\ f'_c k_u bd - f_{su}\rho bd)\ \frac{e'}{d} - \ 1 - \frac{h}{2d}\ \ d \qquad \text{(A18)}$$

The f_{su} , steel stress at ultimate load, can be expressed as

$$f_{su} = E_s \varepsilon_s \qquad \text{(A19)}$$

Referring to Figure A3, it can be shown that

$$\varepsilon_s = \frac{\varepsilon_M(1 - k_u) - \varepsilon_1}{k_u} \qquad \text{(A20)}$$

where

$$\varepsilon_1 = \varepsilon_M(1 - \beta_1)$$

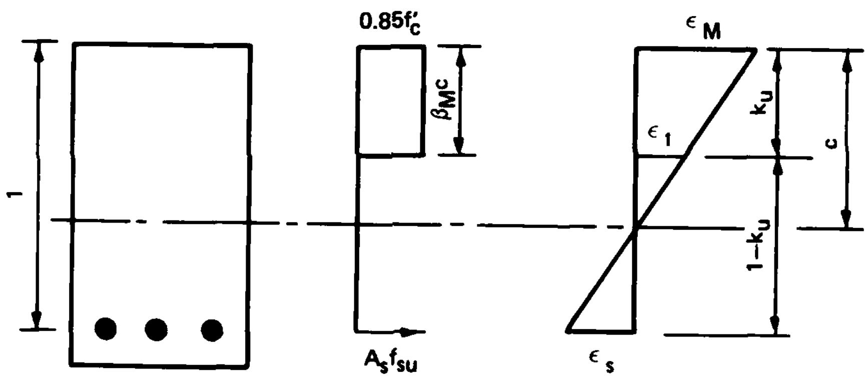

Figure A3. Stress and strain distributions for compression controlled case

Substituting Equation A20 into Equation A19, results in

$$f_{su} = \frac{E_s \varepsilon_M(\beta_M - k_u)}{k_u} \qquad \text{(A21)}$$

Substituting Equation A2 and Equation A21 into Equation A5 gives

$$0.85\ f'_c k_u bde' - \left[\frac{E_s \varepsilon_M (\beta_M - k_u)}{k_u}\right] \rho bde' = 0.425\ f'_c \left(2k_u - k_u^2\right) bd^2 \qquad \text{(A22)}$$

Equation A22 can be rearranged to be:

$$k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2 + \left(\frac{E_s \varepsilon_M \rho e'}{0.425\ f'_c d}\right) k_u - \left(\frac{\beta_M E_s \varepsilon_M}{0.425\ f'_c}\right)\left(\frac{\rho e'}{d}\right) = 0 \qquad \text{(A23)}$$

51. <u>Flexural and compression capacity--tension and compression reinforcement.</u>

<u>a</u>. The design axial load strength ϕP_n is limited to 70 percent of the axial load stength at zero eccentricity and may be computed by

$$\phi P_n = 0.70\ \phi\{0.85\ f'_c[A_g - (\rho + \rho')bd] + E_s \varepsilon_M (\rho + \rho')bd\} \qquad \text{(A24)}$$

<u>b</u>. For doubly reinforced members subjected to combined flexure and compressive axial load, the equilibrium between external and internal forces and moments can be written as

$$\frac{P_u}{\phi} = 0.85\ f'_c k_u bd + f'_{su} \rho' bd - f_{su} \rho bd \qquad \text{(A25)}$$

$$\frac{P_u e'}{\phi} = 0.425\ f'_c \left(2k_u - k_u^2\right) bd^2 + f'_{su} \rho' bd(d - d') \qquad \text{(A26)}$$

The stress-strain distributions at balanced condition are shown in Figure A4. From the strain diagram, it can be shown that

$$k_b = \frac{\beta_M E_s \varepsilon_M}{E_s \varepsilon_M + f_y} \tag{A27}$$

and

$$f'_{su} = \frac{E_s \varepsilon_M \left[k_b - \beta_M\left(\frac{d'}{d}\right)\right]}{k_b} \tag{A28}$$

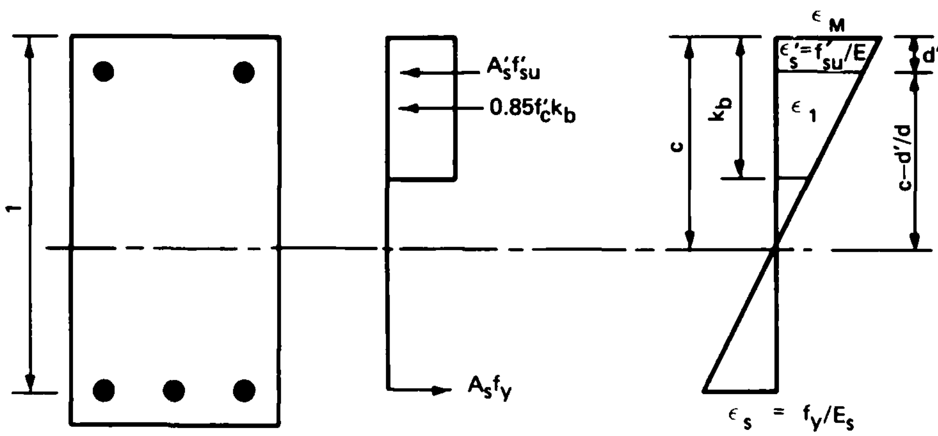

Figure A4. Stress-strain distributions at balanced condition--doubly reinforced member

The e'_b can be determined as follows.

$$e'_b = \frac{P_b e'}{P_b}$$

$$= \frac{0.425\ f'_c\left(2k_b - k_b^2\right)bd^2 + f'_{su}\rho' bd(d - d')}{0.85\ f'_c k_b bd + f'_{su}\rho' bd - f'_{su}\rho bd}$$

$$= \frac{\left(2k_b - k_b^2\right)d + \dfrac{f'_{su}\rho'}{0.425\ f'_c}\ (d - d')}{2k_b + \dfrac{f'_{su}\rho'}{0.425\ f'_c} - \dfrac{f_y\rho}{0.425\ f'_c}}$$

Therefore,

$$\frac{e_b'}{d} = \frac{2k_b - k_b^2 + \dfrac{f_{su}'\rho'\left(1 - \dfrac{d'}{d}\right)}{0.425\ f_c'}}{2k_b - \dfrac{f_y\rho}{0.425\ f_c'} + \dfrac{f_{su}'\rho'}{0.425\ f_c'}} \tag{A29}$$

c. For sections controlled by tension, the stress in the compression reinforcement is small and may be neglected; therefore, Equations 11-13 may also be used for doubly reinforced members.

d. For sections controlled by compression, the ϕP_n can be obtained from Equation A25:

$$\phi P_n = \phi(0.85\ f_c'k_u bd + f_{su}'\rho' bd - f_{su}\rho bd) \tag{A30}$$

The ϕM_n can be obtained by multiplying Equation A30 by e, i.e.,

$$\phi M_n = \phi P_n e$$

$$\phi M_n = \phi(0.85\ f_c'k_u bd + f_{su}'\rho' bd - f_{su}\rho bd)\left[\frac{e'}{d} - \left(1 - \frac{h}{2d}\right)\right]d \tag{A31}$$

where f_{su} is given in Equation A21 and f_{su}' is given in Equation A28 with $k_b = k_u$. From Equations A25 and A26, it can be shown that

$$0.85\ f_c'bde'k_u + f_{su}'\rho' bde' - f_{su}\rho bde' = 0.425\ f_c'\left(2k_u - k_u^2\right)bd^2 + f_{su}'\rho' bd(d - d') \tag{A32}$$

Substituting Equation A21 and Equation A28 with $k_b = k_u$

into Equation A32 and rearranging gives

$$
\begin{aligned}
& k_u^3 + 2\left(\frac{e'}{d} - 1\right) k_u^2 \\
& \quad + \frac{E_s \varepsilon_M}{0.425\, f'_c} \left[(\rho + \rho')\left(\frac{e'}{d}\right) - \rho'\left(1 - \frac{d'}{d}\right)\right] k_u \qquad \text{(A33)} \\
& \quad - \frac{k_1 E_s \varepsilon_M}{0.425\, f'_c} \left[\rho\left(\frac{d'}{d}\right)\left(\frac{e'}{d} + \frac{d'}{d} - 1\right) + \rho\left(\frac{e'}{d}\right)\right] = 0
\end{aligned}
$$

Flexural and tension capacity

52. Figure A5 shows two cases for members subjected to combined flexure and tension, the difference between them being only the degree of eccentricity of the applied load. The design method will vary according to this eccentricity.

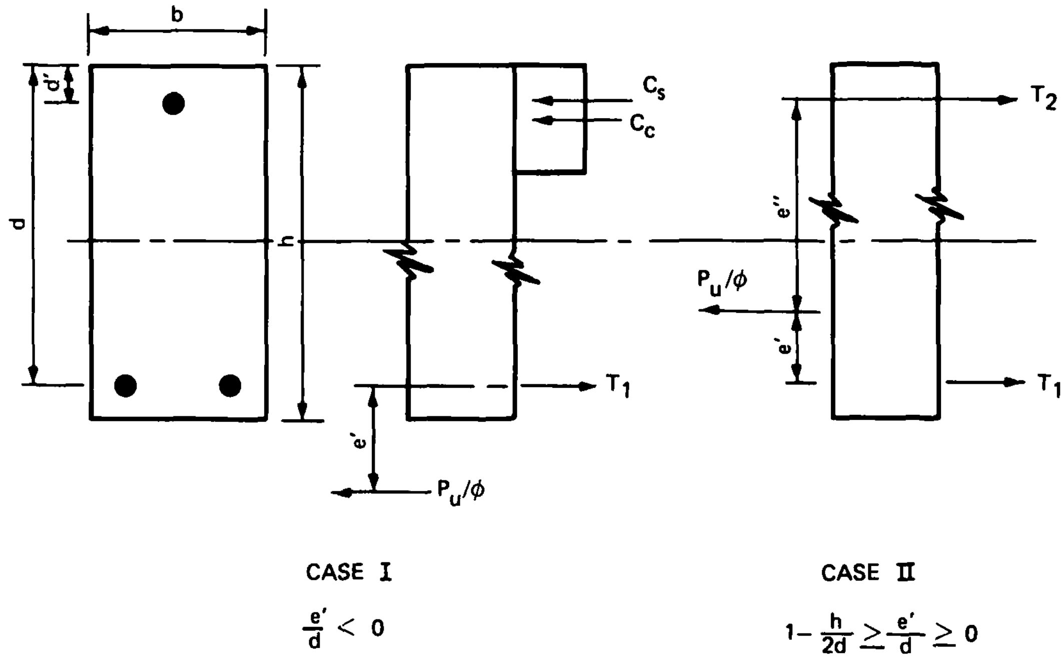

Figure A5. Two cases for members with combined flexure and tension

a. Case I. Case I has eccentricity ratio $e'/d < 0$. It can be seen from Figure A5 that this case is similar to that of the combined flexural and compression case. Therefore, Equations 11-13 are applicable.

b. Case II. In this case, the applied tensile resultant P_u/ϕ lies between the two layers of steel. The forces in the two layers are unequal and are determined by statics. The required steel areas are

$$A_s = \frac{P_u e''}{\phi f_y (d - d')} \tag{A34}$$

$$A'_s = \frac{P_u e'}{\phi f_y (d - d')} \tag{A35}$$

Since $e'' = d - d' - e'$, Equation A34 can be written as

$$A_s = \frac{P_u (d - d' - e')}{\phi f_y (d - d')} \tag{A36}$$

Shear Strength Requirement

Factored shear force

53. Test results indicate that shear strength near supports is increased if compression is introduced into the member. Accordingly, for design stems and toes of T-walls, the critical shear is at a distance d from the support. Since the reaction is in tension in the heels and keys of T-walls and in the stems of L-walls without toes, the factored shear should be computed at the support.

Shear strength of walls

54. The shear provisions of Section 11.3 of ACI 318-77 are specified for T-wall designs. For short keys or toes with a length-to-depth ratio of unit or less, the special provisions for brackets given in Section 11.9 of ACI 318-77 should be followed.

Table A1

Unit Weights and Porosities of Soils*

Type of Soil	Dry Weight lb/ft^3	Saturated Weight lb/ft^3	Drained Weight lb/ft^3	Submerged Weight lb/ft^3	Porosity percent
Cinders	45	72	70	10	44
Clay, soft, inorganic	--	110	110	48	55
Clay, stiff, inorganic	--	129	129	67	37
Clay, soft, organic	--	92	92	30	70
Sand, mixed-grained, dense	116	135	130	73	30
Sand, mixed-grained, loose	99	124	120	62	40
Sand, uniform, dense	109	130	122	68	34
Sand, uniform, loose	90	118	110	56	46
Silt, loose, inorganic	75	109	109	47	55
Silt, dense, inorganic	95	121	121	59	42
Silt, organic	--	89	89	27	75
Sand-clay mixture	132	145	132	83	20

* (Huntington 1957).

APPENDIX B: EFFECT OF FACTORED LOADS ON BASE REACTIONS AND FORCES AT CRITICAL SECTIONS

1. In order to study the effect of factored loads on base reactions and forces at critical sections, five typical T-walls were analyzed using the newly available computer program, TWDA (U. S. Army, Office, Chief of Engineers 1979, 1980).* The magnitude and distribution of the base reactions induced by the factored loads were compared with those obtained under service load condition. The overall load factors for shear and moment at critical sections were evaluated. Based on the results of this study, the load factors specified in paragraph 28 (main text) are justified.

Structural Data

2. Five T-walls, including three retaining walls and two floodwalls of various dimensions, were investigated. The dimensions of the T-walls are shown in Figures B1-B5 and summarized in Table B1. The properties of the backfill materials and water heights are given in Table B2.

Load Cases

3. The following six load cases were analyzed for each structure:

Load Case 1: $D + F_w + F_p + F_u + H_w + H_p$

Load Case 2: $1.5D + 1.9(F_w + F_p + F_u + H_w + H_p)$

Load Case 3: $1.5(D + F_w + F_p + F_u + H_w) + 2.1H_p$

Load Case 4: $1.5(D + F_w + F_p + F_u) + 1.9(H_w + H_p)$

Load Case 5: $1.5(D + F_w + F_p + F_u) + 2.1(H_w + H_p)$

Load Case 6: $1.5(D + F_w + F_p + F_u + H_w) + 1.9\ H_p$

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* The references cited in this appendix are listed in the references at the end of the main text.

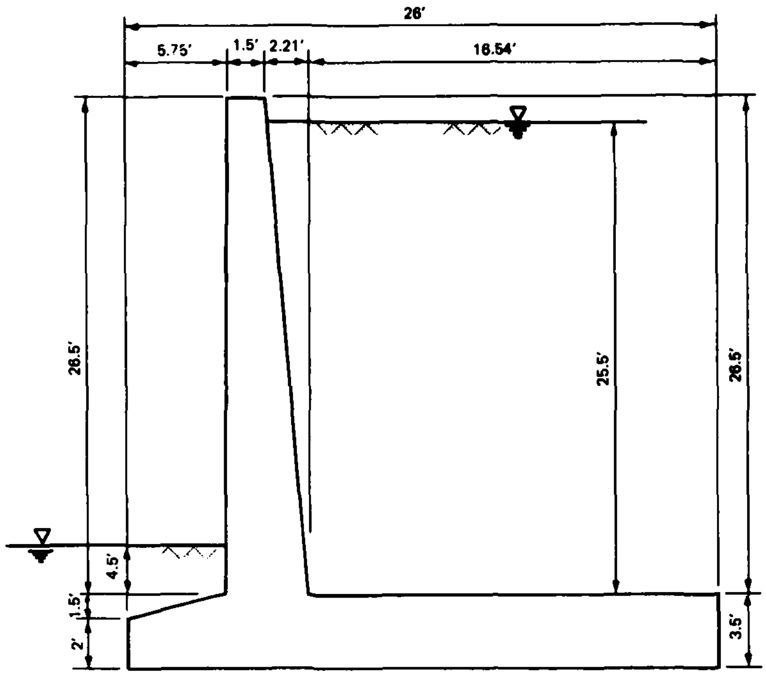

Figure B1. Wall No. 1

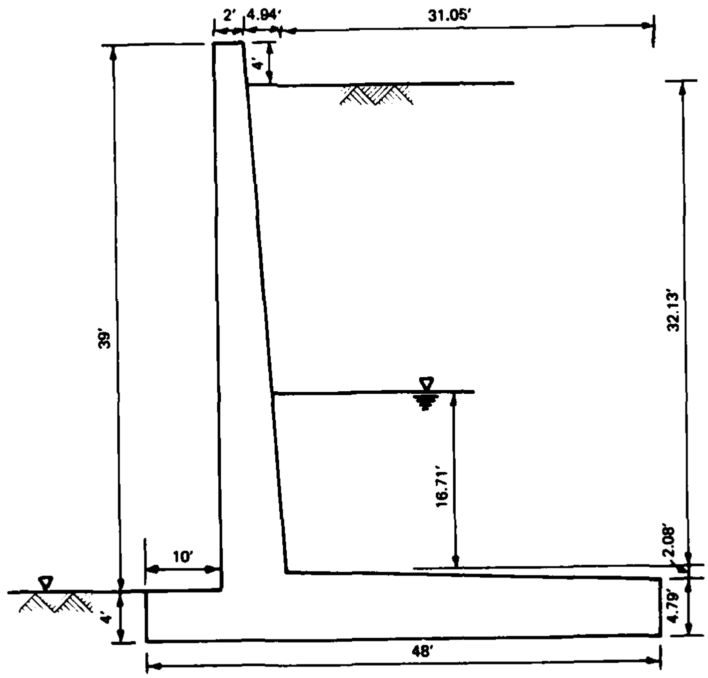

Figure B2. Wall No. 2

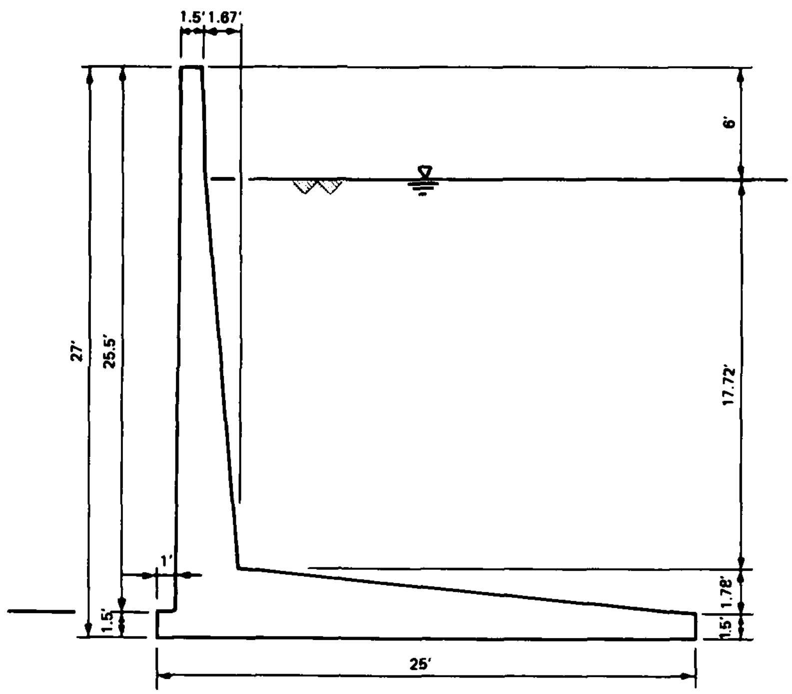

Figure B3. Wall No. 3

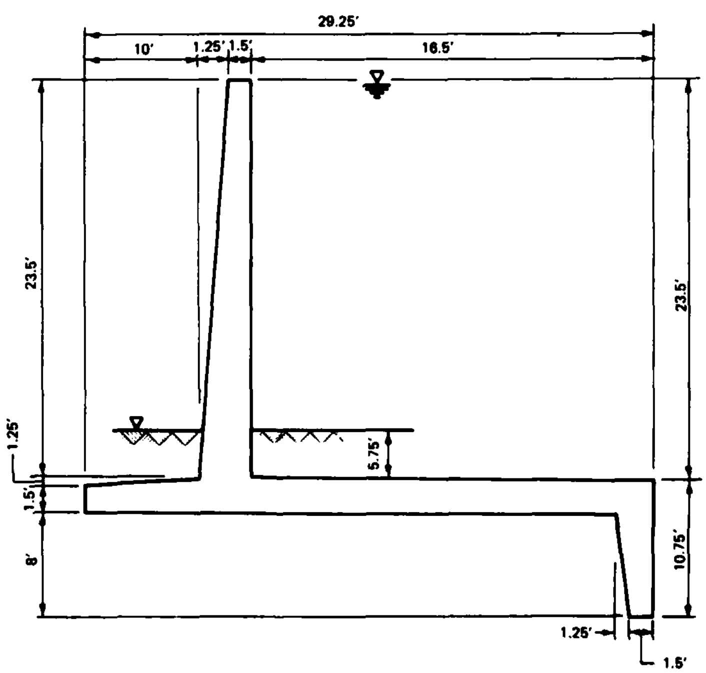

Figure B4. Wall No. 4

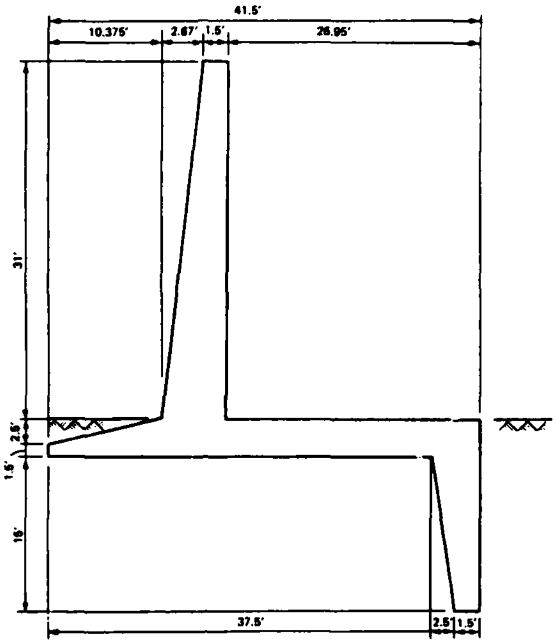

Figure B5. Wall No. 5

4. The load case 1 is a service load condition because the load factor is 1.0 for all loadings. In load case 2, the load factor for a dead load is 1.5, and the load factor for all other loads is 1.9. These factored loads are specified in paragraph 28 in Part II of the main text. The load cases 3-6 were selected to study the effect of factored loads on base reactions and forces at critical sections.

Method of Analysis

5. The structures were analyzed using a computer program, namely TWDA, which is a computer-aided structural design system for analysis and/or design of an inverted-T cantilever wall founded on earth or rock. Earth pressures were calculated by using Coulomb's equations. Hydrostatic pressures were calculated by the line-of-creep method. Since the program in the current version used only the working stress method and factored loads could not be taken into account, modified loading input

was required for factored loads. For example, in load case 2, the unit weight of concrete was increased 1.5 times, and the unit weights of the water and earth were increased 1.9 times to account for the factored loads. In the load cases where the load factors for vertical earth pressure and lateral earth pressure were different (load cases 3 and 6), the K-value for lateral earth pressure was adjusted to account for the difference.

Results

6. The magnitude and distribution of the base reactions induced by the factored loads are summarized in Tables B3-B7. It can be seen that the factored loads (load cases 2-6) only caused slight relocation of the resultant eccentricities from that obtained under service load condition (load case 1). Comparing the locations of the resultant forces between load cases 1 and 2 (service load versus the factored loads specified in paragraph 28 (main text), the deviations were generally less than 1 percent and 9 percent for retaining walls and floodwalls, respectively.

7. Tables B8-B12 show the overall load factors for shear and moment at critical sections. The overall load factors were obtained by dividing the calculated shear and moment by the respective values obtained from load case 1. For the walls investigated, the overall load factors varied from 1.27 to 2.87. In general, higher load factors were found in the heel than those in the toe. However, for the factored loads specified in paragraph 23 of the main text (load case 2), a fairly uniform load factor for all structural elements of the wall was noticed. For example, in load case 2, the average load factors were 1.9, 1.8, 2.4, and 1.9 for stem, toe, heel, and key, respectively.

8. Based on the above discussion, it can be concluded that the factored loads specified in paragraph 28 (main text) will result in designs comparable to those designed by the working stress method.

Table B1

Pertinent Dimensions of Walls

Wall No.	Wall Type	Total Wall Height ft	Stem Height ft	Total Base Length ft	Heel Length ft	Toe Length ft	Key Depth ft	Stem Thickness ft	Heel Thickness ft	Toe Thickness ft
1	Retaining wall	30	26.5	26	16.54	5.75	0	1.5/3.71	3.5	2/3.5
2	Retaining wall	43	39	48	31.05	10	0	2/6.94	4.79/6.87	4
3	Retaining wall	27	23.72	25	20.83	1	0	1.5/3.17	1.5/3.28	1.5
4	Flood-wall	34.25	23.5	29.25	16.5	10	10.75	1.5/2.75	2.75	1.5/2.75
5	Flood-wall	50	31	41.5	26.95	10.375	19	1.5/4.17	4.0	1.5/4.0

Table B2

Properties of Backfill Materials

Wall No.	Angle of Internal Friction deg	Cohesive Strength	Unit Weight pcf	Backfill Over Heel ft	Water Over Heel ft	Backfill Over Toe ft	Water Over Toe ft
1	30	0	128	25.5	25.5	4.5	4.5
2	28	0	125	32.13	16.71	0	0
3	32	0	130	17.72	17.72	0	0
4	21.8	0	120	5.75	23.5	5.75	5.75
5	30	0	125	0	31	0	0

Table B3

Effects of Factored Loads on Base Reaction--Wall No. 1

Load Case	Resultant Vertical Force Acting on the Base kips	Distance of Resultant Force from Toe ft	Magnitude of Base Pressure at Heel ksf	Magnitude of Base Pressure at Toe ksf
1	60.06	10.81	1.14	3.48
2	104.78	10.82	2.00	6.06
3	90.09	10.22	1.24	5.69
4	114.53	11.38	2.76	6.05
5	126.75	11.58	3.28	6.47
6	90.09	10.41	1.40	5.53

Table B4

Effects of Factored Loads on Base Reaction--Wall No. 2

Load Case	Resultant Vertical Force Acting on the Base kips	Distance of Resultant Force from Toe ft	Magnitude of Base Pressure at Heel ksf	Magnitude of Base Pressure at Toe ksf
1	185.45	22.96	3.36	4.37
2	326.54	23.20	6.12	7.48
3	278.21	21.70	4.13	7.46
4	340.34	23.37	6.54	7.64
5	371.40	23.53	7.29	8.19
6	278.18	22.12	4.44	7.16

Table B5

Effects of Factored Loads on Base Reaction--Wall No. 3

Load Case	Resultant Vertical Force Acting on the Base kips	Distance of Resultant Force from Toe ft	Magnitude of Base Pressure at Heel ksf	Magnitude of Base Pressure at Toe ksf
1	66.18	10.90	3.66	1.63
2	119.31	11.10	6.38	3.16
3	99.27	10.29	6.07	1.87
4	120.45	11.15	6.38	3.26
5	131.04	11.24	6.82	3.66
6	99.27	10.50	5.88	2.06

Table B6

Effects of Factored Loads on Base Reaction--Wall No. 4

Load Case	Resultant Vertical Force Acting on the Base kips	Distance of Resultant Force from Toe ft	Magnitude of Base Pressure at Heel ksf	Magnitude of Base Pressure at Toe ksf
1	29.94	12.40	0.33	1.72
2	48.73	11.67	0.31	3.02
3	44.87	12.44	0.51	2.56
4	52.61	12.96	0.77	2.82
5	56.45	13.18	0.91	2.95
6	44.91	12.42	0.51	2.57

Table B7

Effects of Factored Loads on Base Reaction--Wall No. 5

Load Case	Resultant Vertical Force Acting on the Base kips	Distance of Resultant Force from Toe ft	Maximum Pressure at Toe ksf	X-Coordinate of Zero Pressure (Distance from Toe) ft
1	59.40	13.23	3.00	39.67
2	95.94	11.69	5.48	35.01
3	89.11	13.26	4.49	39.73
4	89.76	13.20	4.54	39.52
5	90.08	13.18	4.56	39.53
6	89.11	13.25	4.49	39.73

Table B8

Load Factors for Shear and Moment at Critical Sections--Wall No. 1

Load Case	Stem		Toe		Heel	
	Shear	Moment	Shear	Moment	Shear	Moment
1	1.00	1.00	1.00	1.00	1.00	1.00
2	1.90	1.89	1.77	1.78	2.00	1.94
3	1.71	1.70	1.64	1.63	1.72	1.80
4	1.78	1.77	1.70	1.70	1.91	1.83
5	1.91	1.90	1.79	1.80	2.12	2.00
6	1.64	1.64	1.59	1.59	1.64	1.70

Table B9

Load Factors for Shear and Moment at Critical Sections--Wall No. 2

Load Case	Stem		Toe		Heel	
	Shear	Moment	Shear	Moment	Shear	Moment
1	1.00	1.00	1.00	1.00	1.00	1.00
2	1.90	1.88	1.77	1.77	2.05	1.99
3	2.06	2.03	1.69	1.69	2.03	2.32
4	1.90	1.87	1.78	1.78	2.06	1.99
5	2.09	2.06	1.91	1.91	2.35	2.24
6	1.87	1.86	1.62	1.63	1.86	2.07

Table B10

Load Factors for Shear and Moment at Critical Sections--Wall No. 3

Load Case	Stem		Heel	
	Shear	Moment	Shear	Moment
1	1.00	1.00	1.00	1.00
2	1.90	1.89	2.40	1.88
3	2.10	2.08	2.14	2.16
4	1.90	1.89	2.41	1.89
5	2.10	2.08	2.87	2.09
6	1.90	1.89	1.92	1.94

Table B11

Load Factors for Shear and Moment at Critical Sections--Wall No. 4

Load	Stem		Toe		Key		Heel	
Case	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment*
1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2	1.90	1.90	1.79	1.79	1.90	1.91	2.42	2.35
3	1.50	1.50	1.50	1.49	1.69	1.78	1.49	1.64
4	1.50	1.50	1.46	1.45	1.34	1.09	1.53	1.74
5	1.50	1.50	1.43	1.43	1.27	0.88	1.55	1.86
6	1.50	1.50	1.50	1.50	1.63	1.69	1.49	1.59

* Alternate load case analysis without passive pressure.

Table B12

Load Factors for Shear and Moment at Critical Sections--Wall No. 5

Load	Stem		Toe		Key		Heel	
Case	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment*
1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2	1.90	1.90	1.86	1.85	1.90	1.91	2.30	2.06
3	1.50	1.50	1.50	1.50	1.50	1.49	1.49	1.52
4	1.50	1.50	1.49	1.49	1.48	1.47	1.49	1.54
5	1.50	1.50	1.48	1.48	1.46	1.45	1.48	1.56
6	1.50	1.50	1.50	1.50	1.50	1.49	1.50	1.52

* Alternate load case analysis without passive pressure.

APPENDIX C: DESIGN METHODS FOR CRACK CONTROL

Introduction

1. In the past, flexural cracking in reinforced concrete hydraulic structures has not been a serious practical problem since the stresses in the steel under working loads have been fairly low. However, with the use of higher strength steels, some visible cracks might be expected even under service loads. The resulting cracks are not serious from the point of view of safety; however, they may impair the appearance of a member or increase the risk of corrosion of the reinforcement. In order to avoid this, specific consideration must be given to control of cracking in the design of reinforced concrete members.

2. The objective of this appendix is to review the subject of crack control and its relation to corrosion, with particular reference to the service conditions of concrete hydraulic structures.

3. Some general background information on crack mechanism will first be reviewed. An appraisal of the available field exposure test data relevant to crack widths and corrosion will be made. The existing design methods of controlling crack widths will be evaluated and a recommended method for crack control of reinforced concrete hydraulic structures will be formulated.

Cracking Mechanism

4. Most of the early studies on crack mechansim were based on cracking patterns as observed at the surface of reinforced concrete members and have proposed or implied a cracking mechanism on the assumption that the concrete tensile stress is uniformly distributed over an effective area of concrete (RILEM 1957, Reis et al. 1965),* and the cracks are parallel-sided.

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* The references cited in this appendix are listed in the references at the end of the main text.

5. In 1965, Broms (1965a, b, and c) attempted to define the shape of the internal cracking of reinforced concrete through the injection of epoxy resin into the cracks that occur within loaded specimens. The internal crack pattern of a 6- by 12-in. concrete cylinder reinforced with a single No. 8, high strength bar that was loaded to a nominal stress of 88,700 psi is shown in Figure C1. Two cracks did not extend

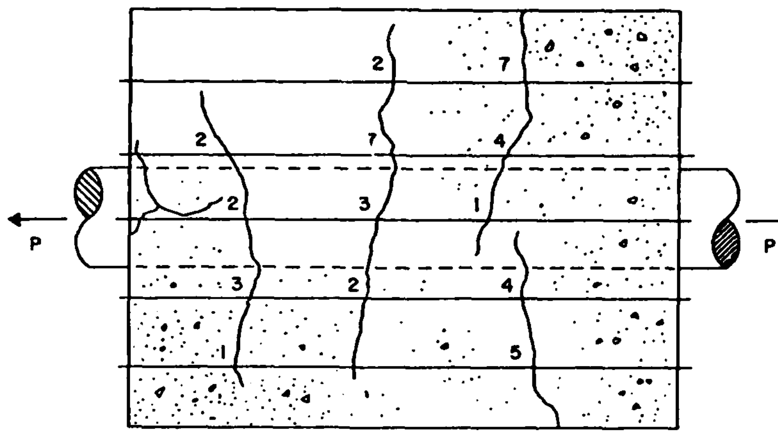

Figure C1. Cracking pattern (Broms 1965c)

to the surface of the concrete cylinder. The maximum width of these internal cracks occurred near the reinforcement. Two other cracks originated near the reinforcement and extended to the surface. The maximum width of these surface cracks occurred at the surface. Additional tests on two rectangular sections with a single reinforcing bar indicated that cracks that did not extend to the surface had a maximum width near the reinforcement and tapered and closed within the concrete, while cracks that reached the surface had a maximum width at the surface and decreased to one-third to one-fifth the maximum width at the level of the reinforcement. Based on experimental studies (Broms 1965 a, b), Broms proposed a cracking mechanism for reinforced concrete members that is based on an elastic analysis of concrete stresses, which is a function of lateral dimensions and crack spacing, and a redistribution of concrete stresses that occurs when a new crack formation alters the geometry of the member.

6. Husain and Ferguson (1968) studied internal flexural cracks in

reinforced concrete beams by filling them with resin while loaded and the load being held constant until the resin had set and then the beam sawed open. Figure C2 gives the crack profiles for various concrete covers and steel stresses. It is interesting to see that though the surface crack width increases with increasing cover, the crack widths at the bar remain roughly constant for a given steel stress.

7. Illston and Steven's (1972) tests confirm the picture provided by Husain and Ferguson but add an important bit of information, namely, that the resin was found to penetrate along the surface of the bar for a considerable distance on either side of the crack into which the resin was injected, which indicates a degree of separation between concrete and steel in this region.

8. Goto (1971) tested long, axially reinforced tension specimens with narrow ducts formed parallel to the reinforcing bar. Ink run into these ducts during loading penetrated all the cracks formed under load so that they could then be located after the test by sectioning the specimen. He noted that numerous internal cracks formed around the deformed bars. These cracks form cones with the apexes near bar lugs and with their bases generally directed towards the nearest primary cracks. Goto also found that the ink penetrated along the bar-concrete interface, indicating a loss of bond. Goto stated that the bond between deformed bars and concrete must then depend on the mechanical resistance of lugs and the frictional resistance between concrete and steel at the bar surfaces between lugs. Based on these tests, Goto proposed an idealized picture (Figure C3) of the forces and deformations around a bar. This picture has been used with considerable success to describe the characteristics of the behavior of deformed bars and to give a qualitative explanation of the splitting mode of bond failure.

9. From the various pieces of research mentioned above, the cracking mechanism in reinforced concrete members under external load can be summarized as follows (Beeby 1978).

10. When the tensile strength of the concrete in a flexural or tensile member is exceeded, surface cracks will form. These cracks will commonly extend to a considerable depth too close to the neutral

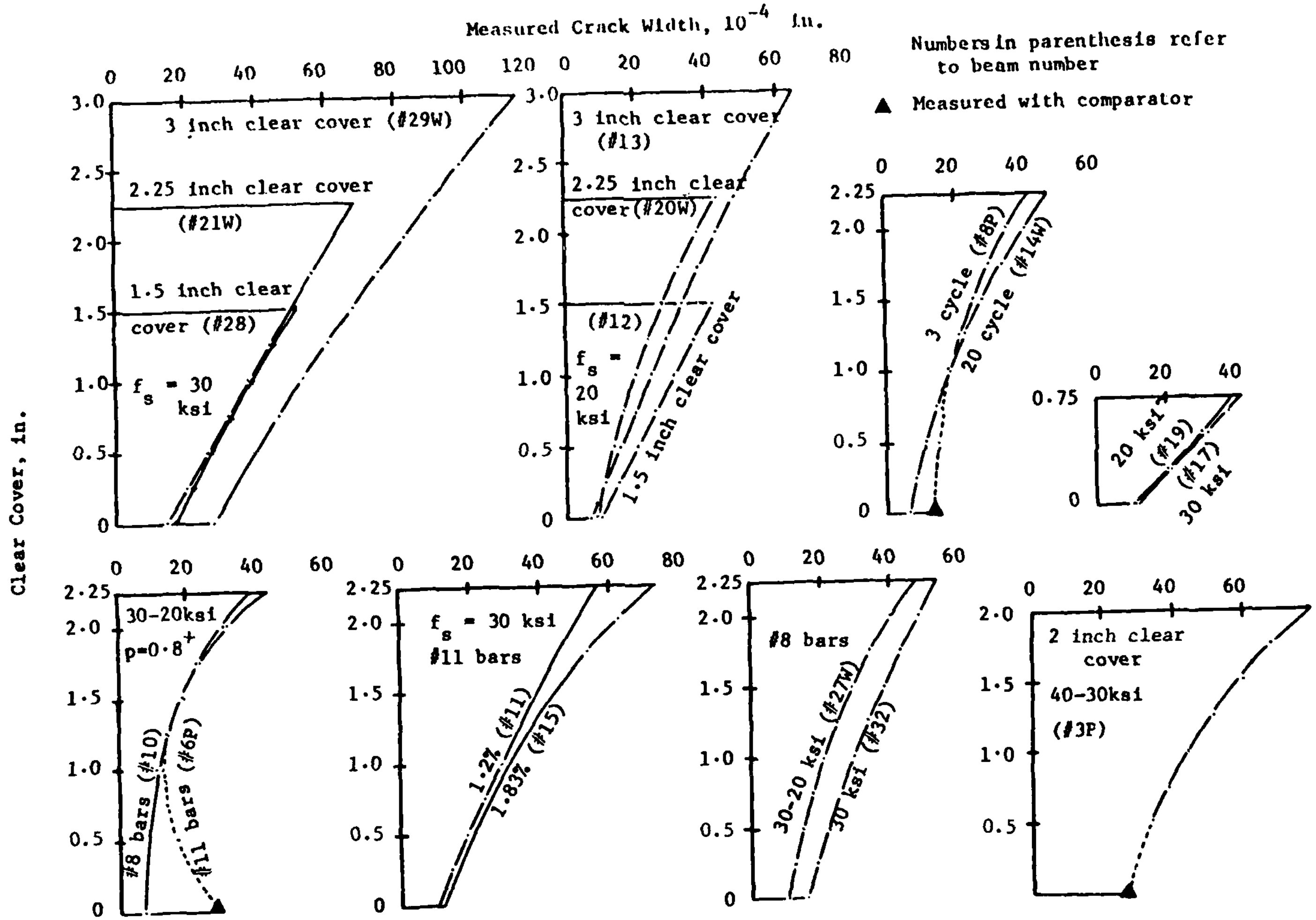

Figure C2. Best fit curves for crack profiles

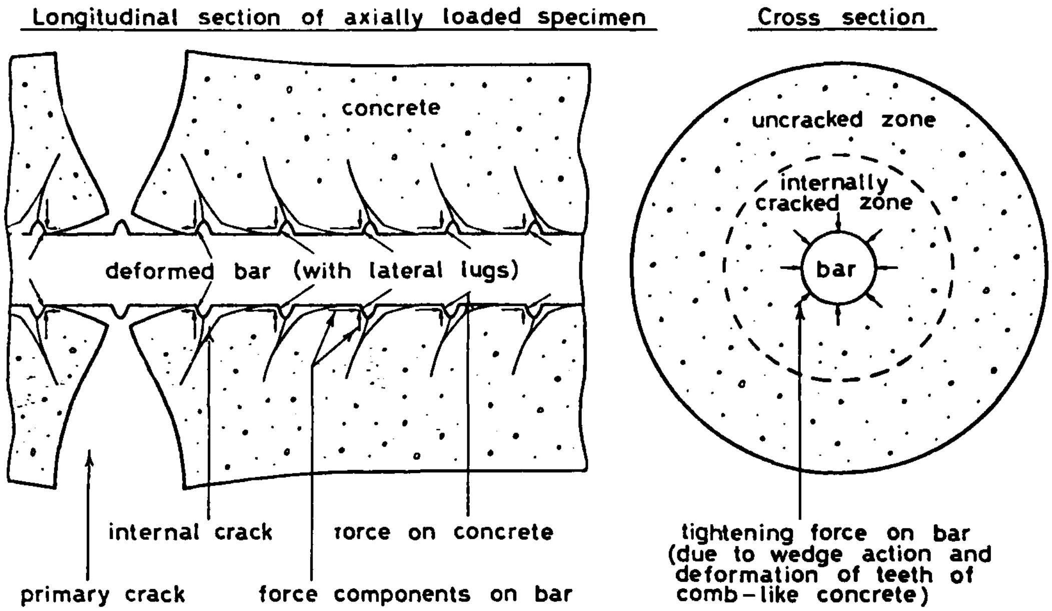

Figure C3. Deformation of concrete around reinforcing steel after formation of internal cracks (schematic diagram)

axis for a flexural member or completely through a tension member. The width of these cracks will be largest at the concrete surface and will, initially, be very small at the bar-concrete interface. While, almost certainly, clear passages will exist between the surface and the steel, any particular section through the crack is likely to appear discontinuous, branched, and irregular. Further loading will cause loss of adhesion along the surface of the bar between ribs, a transfer of force to the ribs adjacent to the main crack, and the initiation of internal cracking from these ribs. As an internal crack develops and extends, the stiffness and hence the ability to carry the load of the 'tooth' or 'cone' of concrete acting on the rib is reduced and the load is transferred back to the next rib. With further loading, internal cracks develop successively further and further from the main crack until the midpoint between two main cracks is reached. Once formed, the main cracks increase in width roughly in proportion to the strain. Even at high strains, they remain irregular, forked, and discontinuous. Their width at the bar surface will be defined by the strains in the bar and the deformation patterns. The width of cracks on the concrete surface depends on a much more complex set of variables though the dominant ones are strain at the surface, cover, and to a lesser degree, steel percentage.

Crack Widths and Corrosion

11. From the structural point of view, probably the most important feature of the corrosion of reinforcement is that the corrosion products occupy a substantially greater volume than that of the steel removed. The result of this is that quite small reductions in sections of the reinforcement will generate sufficient volume of corrosion products to develop substantial bursting forces in the surrounding concrete. Figure C4 shows the bottom concrete cover to the stirrups and main steel of a bridge floor beam that has been completely spalled off due to severe steel corrosion.

12. In this section, the available field exposure test data will be studied, and the relationship between crack widths and corrosion will be evaluated.

Figure C4. Corrosion damage to a bridge floor beam

Tensile crack
exposure tests by WES

13. Two series of reinforced concrete beams were made and exposed to severe natural weathering at Treat Island, Maine, to study the effects of the degree of tension in reinforcing steel and the thickness of the concrete cover on the behavior of reinforced concrete beams. The first series (Series A), installed in 1951, consisted of 82 reinforced concrete beams; the second series (Series B), installed in 1954, consisted of 76 reinforced concrete beams. The variables studied were type of concrete (air-entrained versus nonair-entrained), thickness of concrete cover over reinforcing steel (3/4 in. versus 2 in.), type of reinforcing steel (rail steel versus billet steel), type of deformations of the reinforcing steel (ASTM Standard A 305-50T deformations versus old-style deformations), degree of tensile stress in reinforcing steel (0, 20,000, 30,000, 40,000, and 50,000 psi), and position of steel in the concrete beam at the time of casting (top versus bottom).

14. All beams were 7 ft 9 in. long, while the cross-sectional dimensions were 11-12 in. deep and 8-10 in. wide. They were loaded in pairs as shown in Figure C5. The beams were inspected yearly and and their condition evaluated in terms of a numerical rating ranging

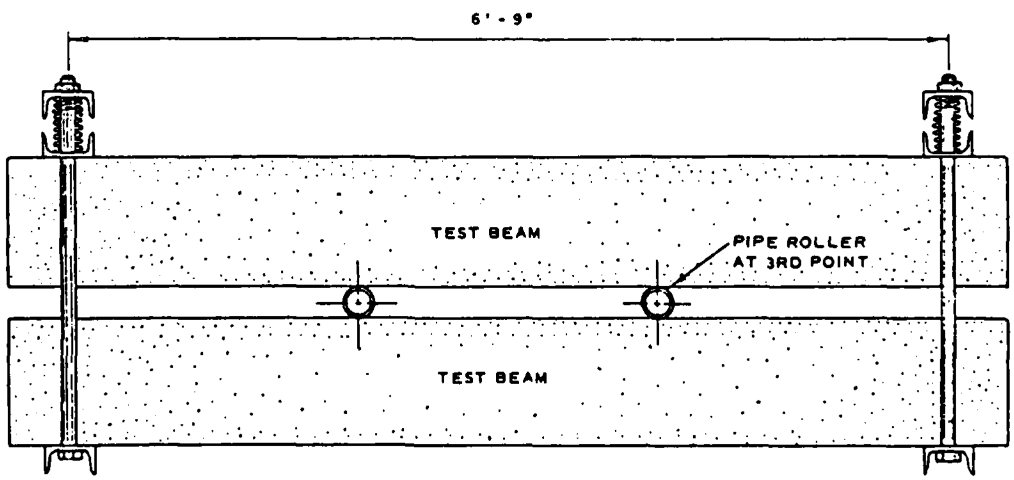

Figure C5. Method of loading tensile crack exposure beams of reinforced concrete from 100 (negligible deterioration) to 0 (complete loss of load-carrying capacity).

15. The first report (Kennedy 1955) on the tests was published in 1955. It describes tests and results up to July 1955. Results after three winters exposure indicated that no correlation was noted between increasing stress and increasing deterioration in the beams of air-entrained concrete. The second report (Roshore 1964) summarizes the results of exposure tests from 1955 to 1963. Results after 12 winters of exposure of the Series A beams indicated that the air-entrained concrete beams were significantly more resistant to weathering than the nonair-entrained beams, and that the beams with reinforcing steel having deformations conforming to ASTM Standard A 305 were more resistant to weathering than those with reinforcing steel having old-style deformations. Test data concerning the other test variables are as yet inconclusive.

16. Finally, O'Neil published a third report (O'Neil 1980) dealing with the laboratory evaluation of the overall performance of the Series A beams after 25 years exposure. The major conclusions were:

- a. Crack widths increased with respect to both time and stress level in the steel, and the resulting crack widths were roughly proportional to the stress level of the steel.
- b. The percent of reduction in the cross-sectional area of the steel due to corrosion did not exhibit any relationship to the stress level during testing. The maximum areas of corrosion occurred at spalled areas, and no areas of maximum reduction of the cross-sectional area occurred at flexural cracks.
- c. The steel was found to be heavily corroded at spalled areas and generally free from corrosion beneath the 3/4-in. cover. Since corrosion could not be found at any cracks in the beams stressed at the 20,000-psi level, it was concluded that crack widths greater than 0.015 in. were necessary to produce corrosion at flexural cracks.
- d. Stressing the steel during the exposure period did not reduce the ultimate moment capacity of the beams below acceptable levels.

Tests by the
Institut fur Massivbau,
Technische Universtat, Munchen

17. Two series of exposure tests have been carried out, each with 10 years duration. The first series was begun in 1957 and the second in 1961. In both series, pairs of identical beams were loaded, as shown in Figure C6, to a level that induced surface crack widths of 0.01-0.02 in. in the central constant moment zone.

18. Sets of specimens were exposed in four different environments:

a. Normal urban environment (Munich).

b. Heavily polluted industrial atmosphere.

c. Tidal zone, North Sea.

d. Marine atmosphere (above tidal zone).

Some of the specimens were broken open to see how much the bars had corroded after 1, 2, 4, and 10 years. The average depth of corrosion was measured at each crack, the width of which was recorded at the start of

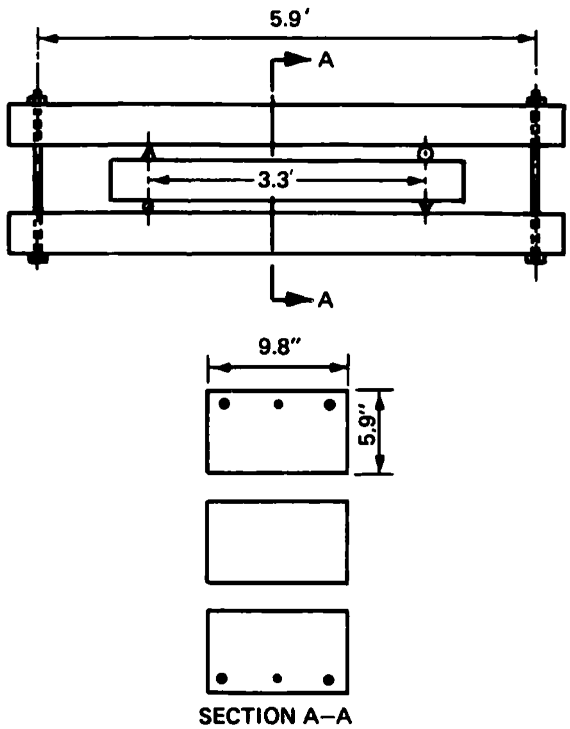

Figure C6. Arrangement of Munich tests

the test and again when the beam was broken open.

19. In 1969, Martin and Schiessl (1969 a, b) published two papers on the tests. These dealt with the results for Series I up to 10 years and Series II up to 4 years. The basic conclusions stated in these papers were:

- a. There is a linear increase, with increase in crack width, in the maximum amounts of corrosion recorded at cracks of a particular size.
- b. For a given crack width, increase in cover gives a substantial decrease in corrosion.
- c. Environment had no significant influence on corrosion.
- d. For equal crack widths, plain bars corroded slightly more than deformed bars.

Tests by Tremper

20. Tremper (1947) tested 64 small specimens each containing a single crack of specified size. These specimens were exposed for 10 years: 2 years on the shore of Puget Sound and 8 in the laboratory yard. Tremper stated that the proximity of the specimens to salt water probably did not contribute significantly to the corrosiveness of the environment. Figure C7 illustrates the specimens used; each contained three bars placed centrally within the blocks. The cracks were produced by flexure.

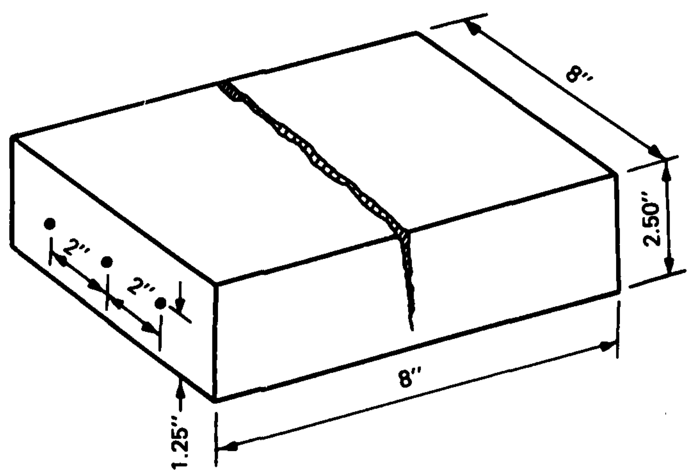

Figure C7. Test specimen used by Tremper (1947)

21. The following variables were investigated:

   a. Type of reinforcement. Three types were used; 16-gauge annealed wire, 7-gauge, cold-drawn wire, 1/4-in. square deformed bar.

   b. Aggregate grading. A well graded and a poorly graded aggregate were compared.

   c. Mixture proportions. Six mixture proportions were used.

   d. Crack widths. Specimens were made with cracks of 0.005, 0.0098, 0.02, and 0.049 in.

22. At the end of the test period the specimens were broken open and the depth of corrosion and the corroded length measured at each crack. In all cases, corrosion was found to be limited to a small area around the crack. The 16-gauge annealed wires were found to be corroded deeply, the 7-gauge cold-drawn, wires were rusted slightly with occasional pits, and the 1/4-in. square deformed bars showed only light rust with no measurable pitting.

23. Tremper found that the average depth of corrosion increases marginally with crack width, though this increase did not appear to be statistically significant, nor did there appear to be any statistical influence of mixture proportions. There were, however, clearly significant differences between the corrosion behavior of the three types of steel used.

Tests by Atimtay and Ferguson

24. Atimtay and Ferguson (1973) report tests on beams and slabs subjected to daily spraying with 3-percent salt solution for up to 2 years. The types of specimen used are illustrated in Figure C8.

25. The variables considered were:

   a. Cover. This was varied from 0.79-1.97 in.

   b. Steel stress and cracking. Unstressed specimens together with beams loaded to give steel stresses of 18.8, 30.0, 36.0 ksi were tested.

   c. Mixture proportions. Concrete strengths were nominally 5.08 ksi, but mixtures were used with water/cement ratios of 0.49, 0.55, and 0.62.

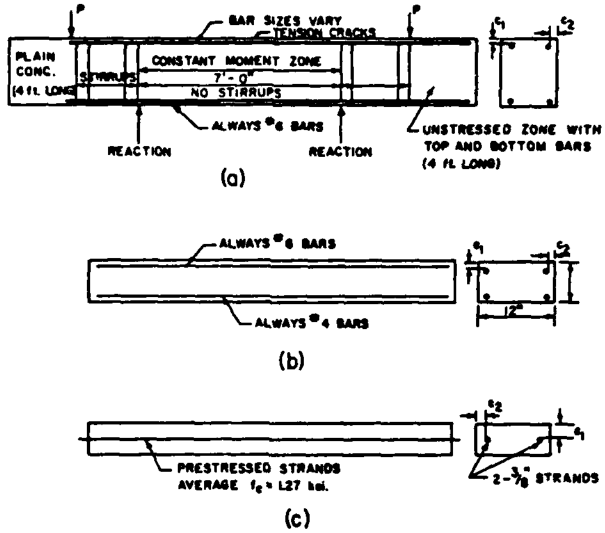

Figure C8. Details of specimens used by Atimtay and Ferguson (1973): (a) flexural beams; (b) unloaded reinforced slabs; (c) prestressed slabs

d. Bar diameter. 0.79-, 0.98-, and 1.38-in. bars were used together with 0.39-in. prestressing wire.

e. Position of steel in mold during casting.

26. During the test period, the specimens were observed, and any deterioration was recorded. At the end of the exposure period the beams were broken open, and the percentage of the surface of the bar that had been corroded was measured. These percentages were used as basic data for further treatment.

27. Observation during exposure showed that corrosion progressed very rapidly in cases where 1-in. cover was given to No. 8 bars; by the end of 19 months the concrete had heavily deteriorated with wide longitudinal cracks along the line of the bars, and 100 percent of the bar surfaces were corroded. Less longitudinal cracking and less corrosion were found where there was 1-in. cover to No. 6 bars. Cover of 2 in. behaved much better than that of 1 in.; longitudinal cracking and corrosion decreased with decreasing bar diameter until, with No. 6 bars, almost no corrosion had occurred over the 2-year period.

28. The dominant variables found to influence the percentage of corrosion were the water-cement ratio and the ratio of cover-to-bar diameter. Stress level and cracking were found to have only a minimal influence.

Discussion

29. Based on the available exposure test data, a qualitative description of the relation between cracking and corrosion can be hypothesized as follows:

a. Cracks permit penetration of carbon dioxide, or chlorides, or both locally into the member; the rate of penetration being a function of crack width.

b. Corrosion will start at the cracks and spread for some distance on either side of the cracks. If the cover is sufficiently thick and of adequate quality, corrosion may never occur in areas other than adjacent to the cracks. However, with thin cover or more porous concretes the corrosion will become generalized.

c. The rate of corrosion will not depend upon the crack width but rather upon the ability of oxygen to diffuse through the concrete to the cathodic regions of the bar and upon the electrical resistance of the path through the concrete between anode and cathode.

d. The corrosion products are highly expansive and will generate bursting forces within the concrete depending upon the depth of corrosion, the length of the corroded region, and the bar diameter. If the concrete surrounding the bar is unable to withstand these forces, the cover will split, producing longitudinal cracks along the line of the bar. With thick covers and small bar diameters, it is possible for very extensive corrosion to occur without the concrete splitting.

Current Design Methods for Crack Control

30. The discussion of the exposure test data suggests that the control of crack widths may not, in fact, have a very significant influence on the durability of a reinforced concrete structure. This does not necessarily imply that the limitation of crack widths in structures is a totally useless exercise, as there may be reasons other than corrosion protection for this limitation. For example, in building

structures, large cracks can adversely affect the appearance and lead to maintenance problems. Watertightness may also suffer if large, permanently open cracks are permitted. With hydraulic structures, appearance is less likely to be of concern, but watertightness in severe circumstances could be important. Therefore, in this section, the various methods proposed in various codes and recommendations will be reviewed their applicability, if any, to hydraulic structures will be determined.

American Concrete Institute
(ACI) Building Code (ACI 318-77)

31. The ACI Building Code (ACI 318-77) (1977) specifies that when design yield strength, f_y, for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the quantity Z given by

$$Z = f_s \sqrt[3]{d_c A} \tag{C1}$$

where

f_s = calculated stress in reinforcement at service loads, ksi

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, in.

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, sq in.

does not exceed 175 kips per in. for interior exposure and 145 kips per in. for exterior exposure.

32. The ACI equation is based on the Gergely-Lutz (1968) expression:

$$W = 0.76\ \beta\ f_s \sqrt[3]{d_c A} \tag{C2}$$

in which W is in units of 0.001 in. To simplify practical design, an approximate value of 1.2 is used for β (ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement). The numerical limitations of $Z = 175$ and 145 kips per in. for interior and exterior exposure, respectively, correspond to

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limiting crack widths of 0.016 and 0.013 in.

33. Equation C2 is a totally empirical formula that was obtained by fitting various combinations of parameters to experimental data on maximum crack width (Broms 1956a, Clark 1956, Hognestad 1962, Kaar and Hognestad 1965, Kaar and Mattock 1963, Rusch and Rehm 1963, 1964). The danger with empirical formulas is that it is highly likely they will predict misleading results in situations outside the limits of the data used to develop the formula. Since the member dimensions, reinforcement arrangement, and thickness of concrete covers used in the hydraulic structures are generally beyond the limited range of the data from which the Gergely-Lutz equation was derived, the use of this equation for hydraulic structures cannot be justified.

European Concrete
Committee (CEB) recommendations

34. Crack control recommendations proposed in the European Model Code for Concrete Structures (Comite European-International du Beton 1978) apply to prestressed as well as reinforced concrete and can be summarized as follows.

35. The mean crack width, w_m, in beams is expressed in terms of the mean crack spacing, s_{rm}, such that

$$w_m = \varepsilon_{sm} s_{rm} \tag{C3}$$

where

$$\varepsilon_{sm} = \frac{f_s}{E_s}\left(1 - k \frac{f_{sr}}{f_s}\right)^2 \leq 0.4 \frac{f_s}{E_s} \tag{C4}$$

and represents the average strain in the steel.

f_s = steel stress at the crack

k = bond coefficient, 1.0 for ribbed bars, reflecting influence of load repetitions and load duration

f_{sr} = steel stress at the crack due to forces causing cracking at the tensile strength of concrete

The mean crack spacing is

$$s_{rm} = 2\left(c + \frac{s}{10}\right) + k_2 k_3 \frac{d_b}{\rho_R} \qquad \text{(C5)}$$

where

- c = clear concrete cover
- s = bar spacing, limited to $15d_b$
- k_2 = 0.4 for ribbed bars
- k_3 = depends on the shape of the stress diagram, 0.125 for bending
- ρ_R = A_s/A_t
- A_t = effective area in tension, depending on arrangement of bars and type of external forces; it is limited by a line $c + 7d_b$ from the tension face for beams; in the case of slabs, not more than halfway to the neutral axis

36. A simplified formula can be derived for the mean crack width in beams with ribbed bars.

$$w_m = 0.7 \frac{f_s}{E_s}\left(3c + 0.05 \frac{d_b}{\rho_R}\right) \qquad \text{(C6)}$$

The permissible crack width as calculated by Equation C3 is limited to 0.004 in. at the level of main steel under frequently occurring loads. It should be noted that the CEB approach is to assume that crack control is only necessary to protect the main steel from corrosion. As a consequence, the crack width requires checking only at the level of the main steel.

British Building Code CP 110
(British Standards Institute 1972)

37. The CP 110 specifies that, unless the environment to which the member is exposed is particularly aggressive, a maximum width of crack not exceeding 0.01 in. will be satisfactory. In addition, in very severely aggressive environments, the maximum crack width at the points on the surface of the member that are closest to the reinforcement

should not exceed 0.004 times the nominal cover to the main bars. In order to minimize the amount of calculation that would otherwise be necessary, the requirements contained in CP 110 to prevent the formation of cracks of excessive width are given in the form of simplified rules. If these rules are complied with, the behavior as regards cracking under service loads should be satisfactory, and further calculation is unnecessary. However, the limiting values given by these simplified rules may be disregarded provided that the designer can still show that the calculated 'designed surface crack widths' do not exceed the above limiting values.

38. Simplified rules. For normal conditions of internal or external exposure, the maximum clear spacing between bars in tension should not exceed the following:

$$\text{clear spacing in millimetres} \leq 75{,}000\ \beta_b/f_y \text{ or } 300$$

where β_b is the ratio of the resistance moment provided at midspan to that actually required. The simplified rules cannot be used when the environment is classified as particularly aggressive unless the value adopted for f_y when calculating the moment of resistance does not exceed 43.5 ksi.

39. Analytical method. According to CP 110, the design surface crack width, w, should be calculated by

$$w = \frac{3\ a_{cr} \varepsilon_m}{1 + \dfrac{2(a_{cr} - C_{min})}{h - x}} \tag{C7}$$

where

w = surface crack width

a_{cr} = distance between the point on the surface at which the crack width is being calculated and the face of the nearest longitudinal bar

C_{min} = minimum concrete cover

h = overall depth

x = depth to the neutral axis obtained when calculating ε_1

ε_m = average strain in the member at the level at which the crack width is being calculated, taking into account the stiffening effect of concrete in tension zone

$$\varepsilon_m = \varepsilon_1 - \frac{0.0012\ b_t h(a' - x)}{A_s f_{st}(h - x)}$$

ε_1 = average strain in the member at the level at which the crack width is being calculated

b_t = width of the member

a' = distance from the compressive face to the point being considered

A_s = steel area

f_{st} = steel stress

40. Comparing the experimental results and the calculated crack width using Equation C7 for a series of beams and slabs, Beeby (1978) concluded that the CP 110 equation predicts the design width with adequate precision ($\pm$ 20 percent) for flexural members.

British code for liquid-retaining structures

41. The maximum calculated surface crack widths for liquid-retaining structures are specified as follows.

Exposure Condition	Max. Calculated Surface Crack Width
Exposed to wetting and drying	0.004 in.
Continuous contact with water	0.008 in.
Others	0.012 in.

42. The crack widths may be calculated according to the following formula:

$$w = \frac{4.5\ a_{cr}\varepsilon_m}{1 + 2.5\ \dfrac{a_{cr} - C_{min}}{h - x}} \qquad \text{(C8)}$$

where

$$\varepsilon_m = \varepsilon_1 - \frac{0.7\ b_t h(a' - x)}{A_x(h - x)f_{st}} \times 10^{-3}$$

and all other notations are defined in Equation C7.

43. In addition, the code also specifies that the crack widths may be deemed to be satisfactory if the steel stress under service conditions does not exceed the appropriate value as shown below.

Exposure Condition	Steel Stress Under Service Conditions
Exposure to wetting and drying	14.5 ksi
Continuous contact with water	18.9 ksi

44. The provisions given in the British Code for liquid-retaining structures are more stringent than those given in CP 110 because crack control is more important in water-retaining structures.

FIP Recommendations for the design and construction of concrete sea structures

45. The permissible surface crack width is limited to 0.012 in. In addition, the assessed surface widths of cracks at points nearest the main reinforcement should not, in general, exceed 0.004 times the nominal cover to the main reinforcement. No formula for calculating the surface crack width is given in the FIP Recommendations.

46. Recognizing the fact that thick concrete cover is important for corrosion protection of steel reinforcement, the FIP Recommendations (1974) specify that the concrete cover should not be less than 3 in. for concrete structures in the splash zone.

ACI 350, Sanitary Structures

47. ACI Committee 350 on Sanitary Structures (Klein, Hoffman, and Rice 1979) recommends that the "Z" factors (see Equation C1) given in ACI 318-77 be limited to 115 and 95 kips/in. for normal and severe sanitary exposure, respectively, corresponding to limiting crack widths

of 0.010 and 0.008 in. ACI 350 further recommends that a maximum of 2 in. for clear cover (Figure C9) be used in the ACI 318-77 equation for "Z," whenever specified cover is greater. The increase in computed values of "Z" with added cover simply reflects a trigonometric projection of the surface crack width, with no particular significance upon durability. Added cover in excess of 2 in. has proven its value for long-term protection, and it may be helpful to consider the excess cover simply as

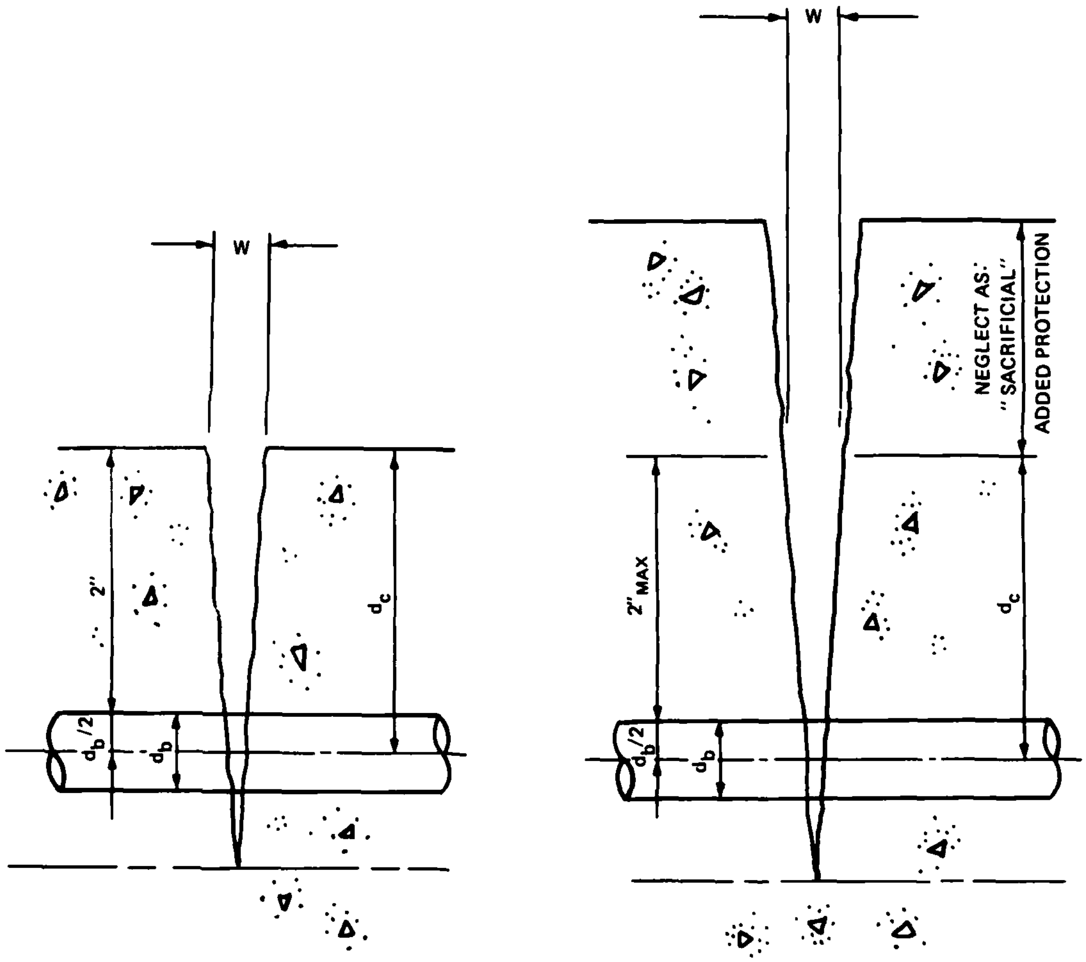

Figure C9. Relationship between crack width and concrete cover "sacrificial" protection against erosion of the concrete to the depth of the steel.

48. In addition to checking the "Z" factor, the maximum bar spacing is limited to 12 in., and the maximum bar size is limited to No. 11.

Australia practice on water-retaining structures (Rangan and Rajasekar 1977)

49. Surface cracks in reinforced concrete structures are limited to a maximum width of 0.004 in. in the case of members exposed to a moist or corrosive atmosphere or where watertightness is essential, and 0.008 in. when members are exposed to continuous or almost continuous contact with liquid. The width of flexure cracks in beams and one-way slabs is controlled by limiting the diameter of reinforcing bars according to the following equation.

$$d_b \left[\frac{1.47}{\rho}\left(1 + \frac{2\ C_{min}}{d_b}\right)\right]^{1/3} \leq \frac{W_{lim}}{\varepsilon_{st}} \qquad \text{(C9)}$$

where

d_b = diameter of bar

ρ = steel ratio

C_{min} = concrete cover

W_{lim} = allowable maximum crack width

ε_{st} = steel strain at service load

50. Equation C9 is derived from Gergely-Lutz (1968) expression (Equation C2) assuming β of 1.2. As discussed before, the Equation C2 is an empirical formula, and its applicability is limited.

Discussion

51. Probably the major difference between the various crack control methods is philosophical. The CEB recommendations require a check on cracking only close to main bars. Cracks elsewhere are not controlled since it is argued that they will not pose a corrosion problem. The British codes for building and water-retaining structures and the FIP

Recommendations attempt to control the maximum cracks anywhere on the member surface largely to avoid visually unacceptable cracks. The ACI practice, both for building and sanitary structures, controls cracking on the tension face of flexural members and not elsewhere. The Australia practice for water-retaining structures is similar to ACI practice. The differences in crack width formulas mainly reflect these differences in what they are intended to predict.

Conclusions

52. Based on the above discussion, the following conclusions may be drawn:

- a. Cracks of the width covered in exposure tests (up to 0.06 in.) will be likely to induce corrosion where bars intersect them, but the amount of corrosion occurring at the cracks over the design life of the structure will not be significantly influenced by the width of the cracks.
- b. Most codes and regulations require either a direct or indirect check on crack width, but these checks, though intended to control possible corrosion, are based on no sound foundation of data relating crack width to corrosion. Nor is there any general agreement as to whether crack widths should be checked or how this check should be carried out.
- c. It appears that a design check on the crack widths, either at points directly over main bars or on the surface, is irrelevant from the point of view of corrosion protection.
- d. With the specified thick concrete cover, low steel stress, small diameter bars and spacings for reinforced concrete hydraulic structures, there is no reason to expect extensive corrosion problems during their design life, regardless of cracking.

APPENDIX D: DERIVATION OF FACTOR β_M

Derivation of General Equations

1. The general concrete compressive stress distribution in a flexural member can be assumed as a parabola (Figure D1):

$$y = Ax^2 + Bx \tag{D1}$$

where

y = concrete stress at distance x from neutral axis

x = distance from neutral axis to the point where stress is considered

A,B = constants (to be determined)

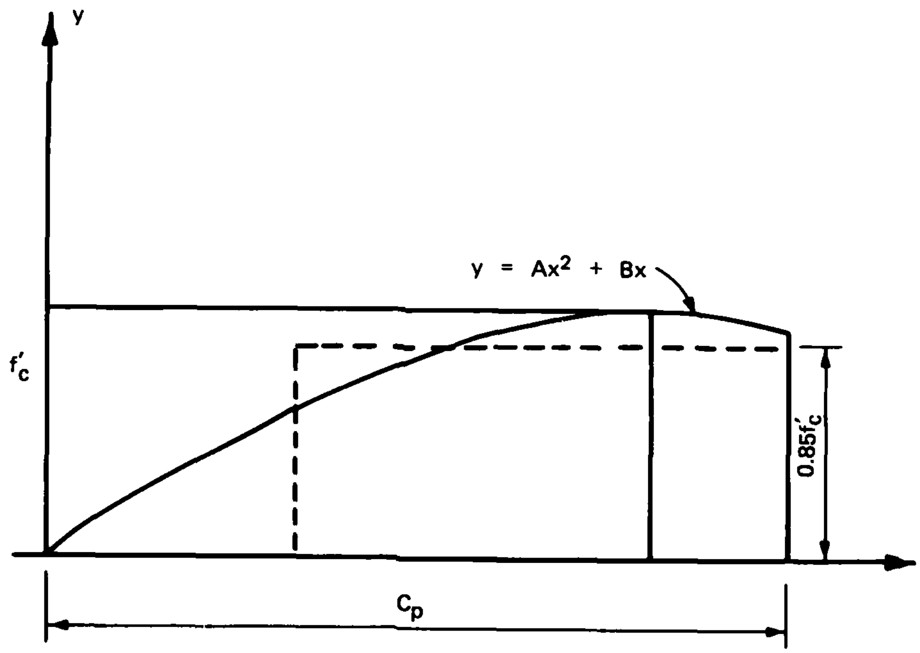

Figure D1. General concrete compressive stress distribution in a flexural member

The area under the parabola, A_p , can be determined as:

$$A_p = \int_o^{C_p} ydx$$

$$= \int_o^{C_p} (Ax^2 + Bx)dx \qquad \text{(D2)}$$

$$= \frac{A}{3} C_p^3 + \frac{B}{2} C_p^2$$

According to ACI 318-77, the actual concrete stress distributions can be approximated by a rectangular stress block. In the equivalent rectangular stress block, an average stress of $0.85\ f'_c$ is used with a rectangle of depth $a = \beta_1 C$. The ACI 318-77 also specifies that the β_1 shall be taken as 0.85 for concrete strengths f'_c up to and including 4000 psi. Thus, A_p can be expressed as

$$A_p = 0.85\ f'_c \times 0.85\ C_p$$

$$= 0.7225\ f'_c C_p \qquad \text{(D3)}$$

for $f'_c \leq 4000$ psi .
Substituting Equation D3 into Equation D2, gives the following

$$0.7225\ f'_c C_p = \frac{A}{3} C_p^3 + \frac{B}{2} C_p^2 \qquad \text{(D4)}$$

Differentiating Equation D1 with respect to x gives

$$\frac{dy}{dx} = 2Ax + B \qquad \text{(D5)}$$

Since there is an extremum point at f'_c , i.e.,

$$\frac{dy}{dx} = 0 \quad \text{at} \quad y = f'_c$$

This condition can be satisfied if

$$2Ax + B = 0 \quad \text{or} \quad x = -\frac{B}{2A} \qquad \text{(D6)}$$

Substituting $y = f'_c$ and Equation D6 into Equation D1 gives

$$\begin{aligned} f'_c &= A\left(-\frac{B}{2A}\right)^2 + B\left(-\frac{B}{2A}\right) \\ &= -\frac{B^2}{4A} \end{aligned} \qquad \text{(D7)}$$

Solving Equations D4 and D7 results in

$$B = \frac{2.4255\ f'_c}{C_p} \qquad \text{(D8)}$$

$$A = -\frac{1.4708\ f'_c}{C_p^2} \qquad \text{(D9)}$$

A general parabolic concrete stress distribution can be obtained by substituting Equations D8 and D9 into Equation D1:

$$y = \frac{1.4708\ f'_c}{C_p}\left(1.649\ x - \frac{x^2}{C_p}\right) \qquad \text{(D10)}$$

The area under the parabola at any point x can be shown to be

$$A_p = \frac{A}{3}x^3 + \frac{B}{2}x^2 \qquad \text{(D11)}$$

After substituting Equations D8 and D9 into Equation D11, the result obtained is a generalized equation for area under the parabola.

$$A_p = \frac{1.4708\ f'_c}{C_p}\left(0.825\ x^2 - \frac{x^3}{3C_p}\right) \qquad \text{(D12)}$$

<u>Derivation of β_M for $\varepsilon_M = 0.0015$</u>

2. With reference to Figure D2, the distance from extreme compression fiber to neutral axis, C, can be computed as follows:

$$C = \frac{0.0015}{0.0015 + 0.0014} = 0.517 \qquad \text{(D13)}$$

and from the similar triangles, the C_p can be determined as:

$$\begin{aligned} C_p &= \left(\frac{0.0030}{0.0015}\right)(0.517) \\ &= 1.035 \end{aligned} \qquad \text{(D14)}$$

The area under the stress distribution curve, A_p, can be computed by

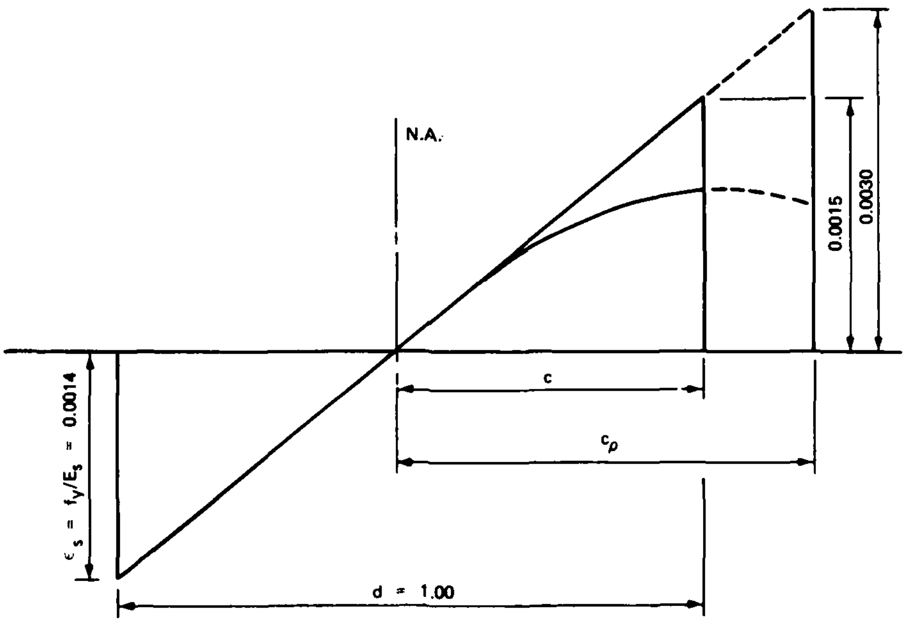

Figure D2. Stress and strain relationship at balanced condition substituting Equations D13 and D14 into Equation D12.

$$A_p = \frac{1.4708\ f'_c}{1.035}\left[(0.825)\ (0.517)^2 - \frac{(0.517)^3}{3\ (1.035)}\right]$$

$$= 0.25\ f'_c$$

The depth of equivalent rectangular stress block, a , can be determined as

$$a = \frac{A_p}{0.85\ f'_c} = \frac{0.25\ f'_c}{0.85\ f'_c} = 0.294 \qquad \text{(D15)}$$

By definition,

$$\beta_M = \frac{a}{c}$$

$$= \frac{0.294}{0.517} = 0.569$$

For design purpose, $\beta_M = 0.55$ will be used.

Derivation of β_M for $f'_c > 4000$ psi

3. The derivations presented in paragraphs 1 and 2 are based on $f'_c \leq 4000$ psi. According to ACI 318-77, for strength above 4000 psi, β_M shall be reduced from 0.85 continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi. Therefore, for $f'_c > 4000$ psi, Equation D3 should be modified. Using the same procedure given in paragraphs 1 and 2, it can be shown that the factor β_M is 0.50 for $f'_c = 5000$ psi.

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Liu, Tony C.
Strength design of reinforced concrete hydraulic structures : Report 3 : T-wall design / by Tony C. Liu (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1982.
105 p. in various pagings : ill. ; 27 cm. -- (Technical report ; SL-80-4, Report 3)
Cover title.
"January 1982."
"Prepared for Office, Chief of Engineers, U.S. Army under CWIS 31623."
Bibliography: p. 42-45.

1. Flood control. 2. Hydraulic structures. 3. Reinforced concrete. 4. Retaining walls. I. United States. Army. Corps of Engineers. Office of the Chief of Engineers. II. U.S. Army Engineer Waterways Experiment Station. Structures Laboratory. III. Title IV. Series:

Liu, Tony C.
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Technical report (U.S. Army Engineer Waterways Experiment Station) ; SL-80-4, Report 3.
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